VESI- JA YMPÄRISTÖHALLITUKSEN MONISTESARJA

Nro 27

SYSTEM ANALYSIS APPLICATION TO DEVELOPMENT OF SCHEMES OF COMPLEX UTILIZATION AND PRESERVATION OF THE WATER RESOURCES

Proceedings of the International Soviet-Finnish Symposium in Water Research in Moscow 10.-14.11.1986

Timo Huttula

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M O N I S T E S A R J A

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PREFACE

According to the agreement of scientific cooperation between USSR Academy of Sciences and the Academy of Sciences of Finland in 1983-1986 joint investigations were carried out within the framework of Project 16. The results of cooperation were presented at a Soviet-Finnish Symposium in Moscow from the 10th to the 14th of November, 1986.

Thirtytwo papers were presented at the symposium. Six papers were presented during the session "Water research in the USSR and Finland". The rest of the papers were discussed in four sessions which were named according to the subjects of the cooperation. Their distribution between the four subjects was a follows: development of optimum management rules for water resources systems (5), development of models of snow cover and snowmelt runoff formation in the forest zone for operative management of water resources systems (5), development of measurements and monitoring of water quality taking into account hydrological factors (10), analysis of regularicties of water quality transformation with the help of hydrodynamic models (6). All of these presentations are not included in this publication. The publication is edited by Timo Huttula.

The discussion of the papers at the symposium showed that, on the whole, Project 16 was carried out succesfully. It was also decided to continue the studies within the Project in 1987-1989. The results will be presented at a new symposium 1989.

On behalf of the Finnish participants I wish to express to the Soviet colleagues and hosts our gratitude and I hope good results from our cooperation.

Helsinki, May 25, 1987

Seppo Mustonen
Leader of the Finnish Delegation

PROGRAMME

OF THE INTERNATIONAL SOVIET-FINNISH SYMPOSIUM ON THE RESULTS OF JOINT RESEARCH WITHIN THE PROJECT 16 "SYSTEM ANALYSIS APPLICATION TO DEVELOPMENT OF SCHEMES OF COMPLEX UTILIZATION AND PRESERVATION OF THE WATER RESOURCES IN LARGE REGIONS IN THE DISTANT FUTURE"

10-16 November, 1986: Moscow, House of Scientists, "White Hall"
(Kropotkinskaya ul., 16)

Monday 10th

10.00	Arrival of the Finnish Scientists to Moscow, (the Leningrad Railway Station)
12.45	Lunch (Moscow House of Scientists, "Blue Hall")
14.30	Opening Ceremony (Moscow House of Scientists, "White Hall")
Section 1.	"WATER RESEARCH IN THE USSR AND FINLAND" Chairmen: G.Voropaev, S. Mustonen
15.00	Study of land water resources in the USSR - G. Voropaev
15.30	Water research in Finland - S. Mustonen
16.00	Modern problems of managing water resource systems and methods of their solution - A. Velikanov
16.30	The need for water research from the viewpoint of one who needs data - M. Raivio
17.00	Departure to the Hotel "Academicheskaya"

Tuesday 11th

9.00	An information system for water resources - R. Lemmelä
9.30	Present status of hydrological research in Finland - E. Kuusisto
Section 2.	"DEVELOPMENT OF OPTIMUM OPERATION RULES FOR WATER RESOURCE SYSTEMS, INCLUDING LAKES AND RESERVOIRS" Chairmen: A. Velikanov, P. Vakkilainen
10.00	Management and flood protection of the Vuoksi River - R. Porttikivi, M. Maunula
10.30	Analysis of water resource system functioning: problems and methods of research - D. Korobova
11.00	The possible use of lakes in the Vuoksi river basin for flood protection of the Lake Saimaa and during a dry spell - M. Maunula

11.30	Break
11.45	Analysis of water resource system functioning in the Vuoksi river basin as the basis for developing manegement rules - D. Korobova, Yu. Oziranskiy, V. Poizner
12.15	The use of long-term forecasts for the non-regulated tributary to the lakes of the Vuoksi basin during the management process - V. Poizner, Yu. Plotkin
12.45	Lunch ("Blue Hall")
Section 3.	"THE DEVELOPMENT OF SNOW COVER FORMATION AND SNOWMELT RUNOFF MODELS IN THE FOREST ZONE FOR OPERATIVE MANAGEMENT OF A WATER RESOURCE SYSTEM" Chairmen: A. Velikanov, P. Vakkilainen
14.00	Snowmelt models in operational use - B. Vehviläinen
14.30	New methods of snow observation in Finland - E. Kuusisto
15.00	Break
15.15	The use of real-time precipitation data in runoff models - P. Vakkilainen
15.45	Developing models of snow cover formation and snowmelt runoff in the forest zone - B. Vehviläinen, Yu. Motovilov
16.15	Discussion of the reports
16.45	Departure to the Hotel "Academicheskaya"
Section 4.	"DEVELOPMENT OF MEASUREMENTS AND MONITORING OF WATER QUALITY TAKING INTO ACCOUNT HYDROLOGICAL FACTORS" Chairmen: M. Khublarian, R. Lemmelä
Wednesday 12th	
9.00	Measurements of very slow currents - J. Sarkkula
9.30	Transformation of natural organic compounds in reservoirs - A. Kocharian, A. Malutin, V. Gekov, I. Lapin
10.00	Collecting and processing of hydrological, water chemical and hydrophysical data - T. Huttula
10.30	A model of structure planning and parameters of water economy, taking into account water protection measures - A. Kocharian, I. Khranovich
11.00	Break
11.15	Phosphorous in bottom sediments of some water bodies and its excretion from bottom to water - M. Martynova, E. Kozlova

11.45	The study of the presence and migration of some heavy metals in bottom sediments for the estimation of secondary pollution scales - N. Grishin, A. Kocharian, A. Malutin
12.15	Efficiency evaluation procedure in setting up discharge-free productions to prevent industrial pollution of natural waters - V. Kaminskiy, I. Orlov, K. Safronova
12.45	Lunch ("The Blue Hall")
14.00	Biomass turnover time and water quality - V. Vavilin, S. Bagotskiy
14.20	Discussion of the raports
Section 5.	"ANALYSIS OF REGULARITIES OF WATER QUALITY TRANSFORMATION WITH THE HELP OF HYDRODYNAMIC MODELS" Chairmen: M. Khublarian, R. Lemmelä
14.50	Intrusion of sea water into coastal freshwater aquifers and its effect on the quality of ground waters - M. Khublarian, A. Frolov
15.20	Mixing of river and sea waters in estuaries - M. Khublarian, A. Frolov
15.50	Transport of dissolved matter in rivers with unsteady flow using St. Venent equations - J. Forsius
16.10	Application of a model describing the phosphorus and nitrogene cycles to Lake Kuortaneenjärvi in the Lapuanjoki basin - J. Kettunen, O. Varis
16.30	Modeling phosphorus transformations in Lake Kuortaneenjärvi ecosystem - A. Leonov, J. Kettunen, O. Varis
16.50	Departure to the Hotel "Academicheskaya"
Thursday 13th	
10.00	Visit of Finnish specialists to the Water Problems Institute: Water Quality Laboratory - A. Kocharian Aquatic Ecosystems Laboratory - A. Vavilin Laboratory of Modeling Hydrological Processes - L. Kuchment Laboratory of Water Resource System Reliability - D. Korobova
12.30	Departure to the Hotel "Academicheskaya"
15.00	Excursion to the Andronnikov Monanstry
Friday 14th	
10.00	Discussion of the reports (The Moscow House of Scientists, "White Hall")
11.00	Discussion of plans for the future cooperation

12.00	Closing ceremony Chairmen: G. Voropaev, M. Khublarian, A. Velikanov, S. Mustonen, P. Vakkilainen. R. Lemmelä
12.45	Lunch ("Blue Hall")
14.30	Excursion to the Museums of Moscow Kremlin
Saturday 15 th 11.00	Excursion to Zagorsk
Sunday 16th 11.00	Excursion about Moscow
20.30	Departure to the Leningradskaya Railway Station from the Hotel "Academicheskaya"

PECULIARITIES OF WATER RESOURCES SYSTEMS MANAGEMENT; SIDE-EFFECTS OF THEIR DEVELOPMENT

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Water, one of the major natural resources, is essential for economic growth and social prosperity. Worldover, industrial development, agricultural production and improvement of social wellbeing constantly increase water use. This concerns both fresh water withdrawls and consumptive water use. According to World Watch paper 65 irrevocable consumptive water withdrawls have increased from 800 km of water in the 1950s to more than 3500 km^3/ year currently [1].

The major water use is irrigated agriculture; the irrigated area worldwide now totals about 270 million hectares, out of which 20 million ha are in the USSR. UNESCO estimates that about 60 % of increment in crop yield annually is provided by irrigated cropland. It is projected to irrigate additional 50 million ha of land in the world by the end of the century.

Compared with irrigated agriculture industrial and municipal water consumption is relatively small. However, fresh water withdrawls for industrial and municipal demands are commensurable with water withdrawls for irrigation. For example, in the USSR in 1984 more than 120 km^3 of water was withdrawn to meet the demands of industrial and municipal water users.

It should be noted, that in recent years a tendency for reduction of demands for water in all economic branches has arisen in industrial countries. In a number of cases a stabilization and even partial decline in the rate of industrial and municipal water use is observed. However, this process leads to an increase of water users demands on water supply reliability, thus imposing new demands on water resources management.

Nowadays water systems play an important role in the solution of transportation problems. Water transport is the most economic and energy-saving type of transportation and for a number of northern areas in the USSR it is the only possible transportation route.

The use of inland water resources for recreation is ever growing. Water tourism, fisheries, sailing and rowing impose serious demands on the hydrologic regime of water bodies. All these aspects entail the development of complex water resources systems [2] - hydraulically connected water sources, systems of hydrotechnical projects, reservoirs and channels for river runoff regulation and water allocation to water users [2]. Like other technical systems (electric power production, transportation, communication, etc.) water resources systems are one of the major elements of the national economy. However, there are some specific feateures, involved in water resources systems: river runoff variability in time, uneven distribution of water in space, and close interaction of water resources systems with the environment. These very pecularities should be considered in the first place to determine methods for water resources systems development and management.

Streamflow fluctuations are stochastic in nature, they are governed bu probabilistic laws. Therefore, long-term streamflow prediction should be done on the pobabalistic basis. Stochastic nature of streamflow fluctuations is one of the important characteristics, which should be considered in the planning of water resrources development, water supply in particular.

In the USSR a reliability index is used for analysing water resources management problems. As a rule, this index is determined on the basis of water management experience. The estimated reliability, as a probabilistic index or criterion of uninterrupted (regular) water supply, is defined separately for various branches of national economy and reflects the reaction of water users to disruptions in water supply.

The estimated reliability indeices used in the USSR may be subdivided for three types of water users [3].

1. Users, requiring practically uninterrupted water supply; their demands are met with a 95 - 99 percent of reliability (e.g. 95 % reliability means that during a period of 100 years deficits occur in 5 years spans).

2. Users that can afford more or less frequent deviations from the water supply regime; an 85 - 90 percent reliability.

3. Users, demanding huge amounts of water per unit of production, that have reserves and use the excess of streamflow most completely; a 75 - 80 percent reliability.

The first type includes municipal water supply, industry and large hydropower plants with long-term streamflow regulation. The second type comprises river transport and small hydropower plants. The third type encompasses agriculture and fisheries.

The estimated reliability of water supply is the most simple and widely used index and is used to determine the parameters of water resource systems.

However, this index does not take into account the water supply regime and, therefore, the working regime of resesrvoirs during water deficit (dry) years is not considered. For this reason another index - admissible depth of reduced water supply in the dificit years - has been introduced into water management practice. For example, the estimated reliability of water supply of irrigated agriculture in the USSR amounts to 90 %, and in 10 deficit years water supplies are not to be curtailed by more than 20 % of the required supplies.

The experience of large water resources systems operation has shown that the reliability indices do not fully incorporate ecological requirements, involved in the use of natural resources.

Irreverible processes in ecosystems are related not only to the number of years and the depth of water supply interruption but also depend on the duration of interruption. Therefore, an additional index of reliability has been introduced, characterising permissible duration of low-water year sequences. At present, this index is not widely used, but in the future with the increase of environmental demands on water resources systems development the role of the index will increase.

Until recently the problem of estimated reliability has been considered separately for each branch of national economy. But the experience and future development of water resources systems management necessitate to analyse this approach. An analysis of water resources systems, comprising water sources and water users, reveals some pecularities. First, there is no direct relationship between water demands of individual user and anticipated water shortage, as the latter is determined not only by the existing variability of flow but also by the existing procedure of water use. In addition, with system components scattered over the river basin and the high differentiation of water-use patters (consumptive or non-consuptive water withdrawls, use of the river natural flopw, etc.), the relationship between aggregate water demands and water shortages, with a definite storage capacity of reservoirs, is uncertain. The reliability of meeting the demands of individual water users is the function not only of runoff regulation, but also of some other factors which are not taken into account by estimated reliabilities for individual branches of economy. In this regard, the problem of estimation of water supply reliability is a very complicated one.

The solution of this problem is closely connected with another pecularity of water resources systems, i.e. their interrelation with the environment. Water resources systems themselves are one of the inherent elements of the environment. Their development and management in the process of functioning causes a number of side effects, the proper account of which is the problem of major importance.

The experience of large water resources systems construction indicates that besides the solution of definite problems, determined in the planning stage, a number of side effects in the environment, economic and social development of the area may occur during implementation stage and, esopecially, in the period of water resources management.

Future ecological and socio-economic impacts that envolve during the stage of water resources systems construction and management and are beyond the objectives of the project are called side effects of water resources development.

Evolution of water development from simple water wheels to ambitious projects of runoff transfer (NEWAPA) [4] entailed a shift of the emphasis in the analysis and evalution of side effects, both positive and negative. In the early stages of water development hydroprojects were single purposem, in the process of their management a possibility arose to use the positive effects to serve economic and social purpose. As a result, the development of multiple purpose reservoirs was initiated. It is evident, that construction of hydroprojects, their integration in cascades, and development of water resources systems allow one to solve not only the direct problems, i.e. to meet the demands of various branches of national economy - municipal, water transportation, power production, industrial, etc., but to facilitater development of an infrastructure, expansion of business activities, developing of new areas. For example, the construction of a cascade of hydroprojects on the rivers Angara and Enisey in the USSR not only provided for the solution of energy and water transportation problems, but formed the backbone of the vast industrial complex [5].

In recent years water projects have been planned to facilitate recreation and tourism, and not long ago provision of recreation facilities was considered to be a side effect of a project. The experience of water resources systems construction in the USSR has shown that their role in the solution of economic and social problems is ever growing. In the process of system's management priorities of alternative water users can be redistributed. All these facts stress that with further development and improvement of water resources systems positive side effects become an integral part of the objective function of the projects. This is relevant to economic and social after-effects, which might occur as a result of water project's implementation.

At the same time, rational and wise development of water resources systems has a positive impact on the environment. The development of a network of channels in arid zones positively modified the flora and fauna of irrigated areaas, improves the microclimate of adjacent territories. Man-made water bodies as are used watering places by wild animals; improve the ecological state of the area. In the zones of excessive precipitation, water projects, mainly channels, drain the adjacent areas, which is favourable for vegatation modification, in particular, the replacement of leaf-bearing forests for confiderous ones.

It should be noted, however, that positive environmental side effects are not yet comprehensively studied. The very notion "positive influence on the environment" is called in question by many scientists, based on one of the concepts of interrelation of Man and the Environment. In accordance with the concept known by the Memorandum of the Roman Club "The Limits to Growth", human activities are anomalous from ecological point of view, they disturb existing natural interrelations in the biosphere [6]. The scientific revolution has signified the beginning of a global ecological crisis, caused by technologic violation of natural equilibrium. The limits on man's activities are determined by the structure of the modern biosphere and by functional relations in it. These limits have been already reached. The concept, applied to the problem of water resources development, leads to a conclusion that water resources development is ecologically harmfull, stresses the role of negative side effects of water resources systems development.

In our opinion, assessment of negative side effects should be based on a concept of modern development of biosphere, worked out by the Sovied Academician V. I. Vernadskii [7]. In accordance with the concept, human activities are the natural and the most comprehensive representation of the main trend of the biospheric evolution. The scientific progress accounts for the transition of the biosphere into highest phase of its development - a noosphere, where scientific thought materialized in the corresponding technologies, controls all process in the biosphere. The limits, imposed on human activities, are determinded by the level of a scientific knowledge and existing socio-economic possibilities to utilize the developed technologies.

Based on this consept the strategy of natural resources use and the development of water resources, in particular, presuppose that the new equilibrium in the biosphere as a whole, and in its separate blocks (hydrosphere), and sub-blocks (fresh water runoff) could be maintained by active interaction of natural and technogenic systems (e.g. water resources systems). We should analysis the negative side effects of water resources systems development from this angle.

Identification and analysis of the negative side effects, occuring during the construction and management of water resources systems, should be aimed at the development of the ways to maintain equilibrium in the noosphere.

The theory and practice of water resources systems construction and management allows one to determine a number of side effects, types of negative effects should be differentiated.

The first type incorporates the impacts, not aimed by the project, which could be identified before its implementation; to prevent their occurance and to mitigate negative impacts countermeasures should be worked out. Such effect and measures to mitigate them can be economically assessed, the water resources systems construction is not sticly constrained by them.

The second type of the negative side effects incorporates implication, which should not occur, to preserve environmental stability. This type of side effects could not be predicated with the necessary rate of reliability. It should be analysed as a system of constraints on the project's parameteres.

In identification of the negative side effects it is necessary to follow up not only the direct impact of project's implementation, but hte whole chain of interactions of the natural and social-economic processes with a water project. It is nesessary to provide a spatial and temporal analysis of side effects.

Defending on the scale of water projects, the levels of spreading of side effects could be outlined:

a) local side effects (with linear dimensions of 10 - 15 km from the construction site);

b) regional side effects (dozens and even hundreds kilometers from the construction site);

c) global side effects (thousands kilometers from the construction site).

Side effects should be analysed for the period of transition in the environment, caused by water resources construction, and also for a fixed regime, predetermining the temporal scale to be used for analysis of side effects. Side effects in hydrologic and hydrochemical processes manifest themselves after relatively small periods of time (days, months, a year); and in hydrologic or climatic processes side effects are manifested after longer time intervals. Side effects in biological processes can manifest themselves in any of the above mentioned intervals of time.

Among the major processes, where side effects, resulting from water resources systems development could surface, are:

- modification of a hydrologic regime, determining changes in hydrophysical and hydrochemical processes both on the construction site, and in the zones of impacts; including estuaries;
- changes in the hydrologic regime of adjacent areas as a result of inudation and drainage of lands by channels and the decrease in water level of water ways in downstream reservoirs;
- modification of aquatic ecosystems, determined by changes in the hydrologic regime;
- modification of terrestrial ecosystems, caused by the development of new water projects and by changes in the hydrologic regime of adjacent areas;
- microclimate changes, occuring as a result of project's implementation or considerable changes in the natural regime of water ways;
- regional climatic changes, conditioned by large-scale water resources systems projects such as interregional runoff transfer;

- socio-economic after-effects of water resources development.

All these processes and changes should be analysed on the stage of a feasibility study and planning of water resources development of assess both positive and negative side effects.

CONCLUSIONS

Water resources systems development imposes a number of scientific problems. The efficiency of water use depends on the successful analysis of these problems. The problems encompass: the development of reliable methods for modelling hydrologic series for a number of sites simultaneously, allowing one to present statistically hydrologic state in the future; the development of simulation modelling of water resources systems functioning to obtain reliable hydrologic and water resources characteristics or indices and to develop operating policies (rules) for complex water resources systems; improvement and wide introduction of physical-mathematical modelling for the solution of operating problems in real time and to increase the correctness of hydrologic data used in water resources planning and projecting; assessement of water balance elements for large reservoirs.

The management of large water resources systems on the stage of functioning is one of the major national economic problems, influencing agricultural productivity, energy production, river transport. Therefore, a desire to use short-term and long-term hydrometeorologic forecasts to increase reliability of operation arises. At the same time, we should not forget that the regularities of natural streamflow fluctuations identified with the help of static data are the single reliable basis for an assessement of the guaranteed water resources yield. Water accumulation in reservoirs to get the estimated (with the definite rate of probability) water supply of national economy should be the main principle of operation.

Of utmost importance are the ecological demands on the water resources systems management, water quality included. At present, there is an evident disproportion between the levels of hydrologic and ecological substantiation of water resources systewms management. For an improved basis of ecological and other environmental demands on the regime of water resources systems performance a systems approach should be used, i.e. the joint efforts of hydrologists, hydraulic engineers, ecologists and economists.

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PRESENT STATUS OF HYDROLOGICAL RESEARCH IN FINLAND

Hydrological research in Finland faces some new important tasks. The acidification of soil, groundwater and lakes will inevitably cause problems in a near future. These problems cannot be solved without detailed studies on the movement of water within the soil and on water exchange in lakes.

Better flood forecasts are needed particularly in river basins of Ostrobothnia and northern Finland. A particular attention should be focused on the modelling of ice phenomena during winter and spring floods.

The use of remote sensing in hydrology will increase. Gamma-ray spectrometry and satellite pictures have aleady been used in snow studies. Satellite pictures are also useful in observing freezing and break-up dates and surface water temperature in lakes.

MANAGEMENT AND FLOOD CONTROL OF THE SYSTEM OF RIVER VUOKSI

R. Porttikivi and M. Maunula

The Vuoksi watercourse is a transboundary watercourse for Finland and the USSR, as the upper reaches and main part of River Vuoksi are in the territory of the USSR (appendix 1). The development of the watercourse, flood control and protection of the watercourse are common goals for both countries. Joint affairs concerning the watercourse are dealt with in the joint Finnish - Soviet boundary water commission.

The number of lakes in the system of River Vuoksi is very high. Floods and droughts occur seldom because the climatic conditions are stable. Numerous flow regulations have been implemented in the watercourse. The largest regulated lakes of the Vuoksi river system are Kallavesi - Unnukka (1 000 km^2), Höytiäinen (290 km^2) and Koitere (180 km^2). Of the large lakes, Lake Saimaa and Lake Pielinen are in a natural state (appendix 2).

In the Vuoksi watercourse, the mean annual precipitation is 590 mm and evaporation 405 mm. Normally the water equivalent of snow is at its greatest at the beginning of April. The average maximum is 150 mm.

The variation in water level of Lake Saimaa in its natural state is unusually high among Finnish lakes, exceeding 3 m (Fig. 1). The mean high water level is, however, only 0,70 m higher than the mean low water level. The mean discharge of the Vuoksi River is 590 m^3/s, maximum 1 190 m^3/s (in 1899), and minimum 220 m^3/s (in 1942).

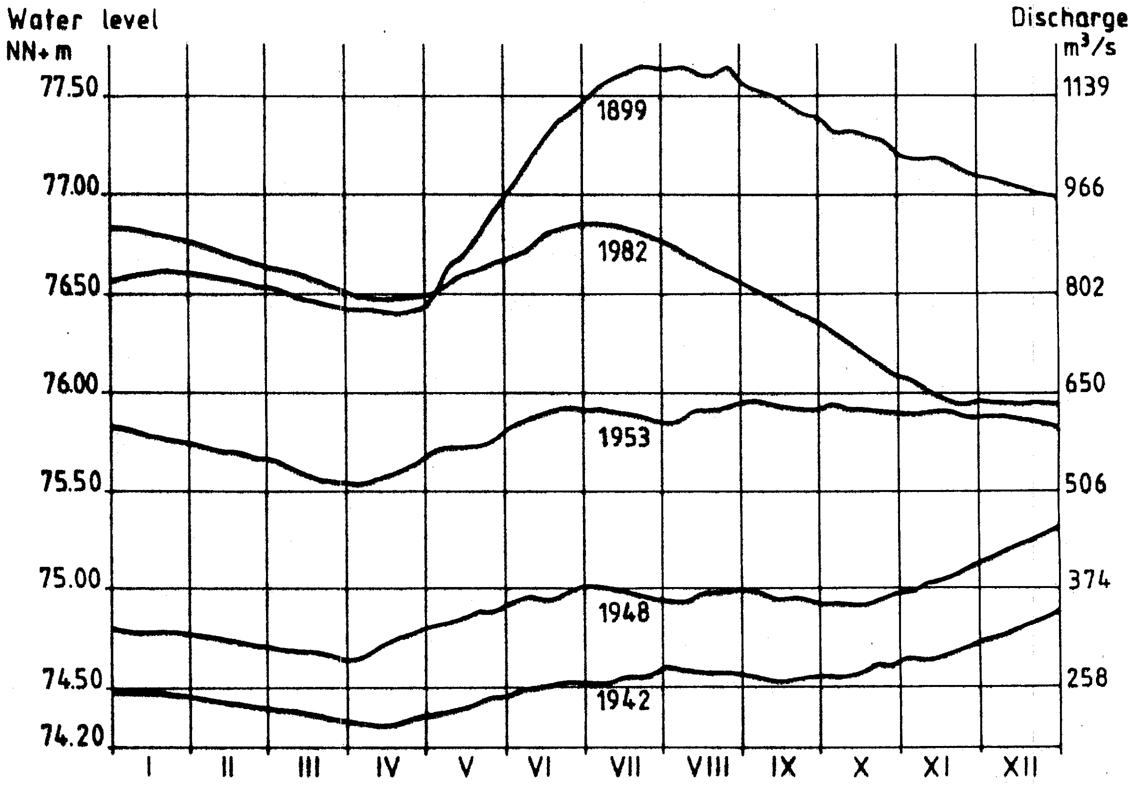


Fig. 1 Natural water elevations and discharges of Lake Saimaa in some years.

The upstream areas of Vuoksi watercourse are hydrologically quite different from the area surrounding Lake Saimaa. Due to the great number of large lakes, the time lag of the flood peak is about 2 months between the upstream areas and Lake Saimaa. In Lake Saimaa the water level reaches its peak not earlier than in the middle of July, whereas the peak in the upper reaches occurs in May.

The Vuoksi water system is used in many different ways. More timber is floated in the Vuoksi river system than in any other inland watercourse, and an important part of Finnish pulp and paper industry is situated in the area. Of great significance for the development of water traffic was the reopening in 1968 of the Saimaa canal which connects Lake Saimaa with Gulf of Finland. The hydropower plants of the area produce annually about 2 TWh or nearly 20 % of Finlands total hydropower production. The total production of Tainionkoski and Imatra power plants, situated in the Finnish part of the Vuoksi river, is about 1,3 TWh, which is the same as the total production of the Svetogorsk and Lesogorsk power plants in the Soviet part.

The Vuoksi water system is also important for recreation.

Floods cause damages much more often than droughts because of the climatic conditions in Finland and the great amount of lakes. When water level is rising, the flooding first damages fields (Fig. 2). After that, different kinds of houses, roads and forests are in danger. In addition, Lake Saimaa is surrounded by a considerable amount of factories (mainly paper mills) which suffer remarkably from flood when it is at its worst. On the other hand, flood also causes remarkable damages on the Soviet side, when discharge in River Vuoksi is great. Therefore, the threat of damages and the impact of various actions in both countries have to be taken into consideration when planning and executing flood protection actions.

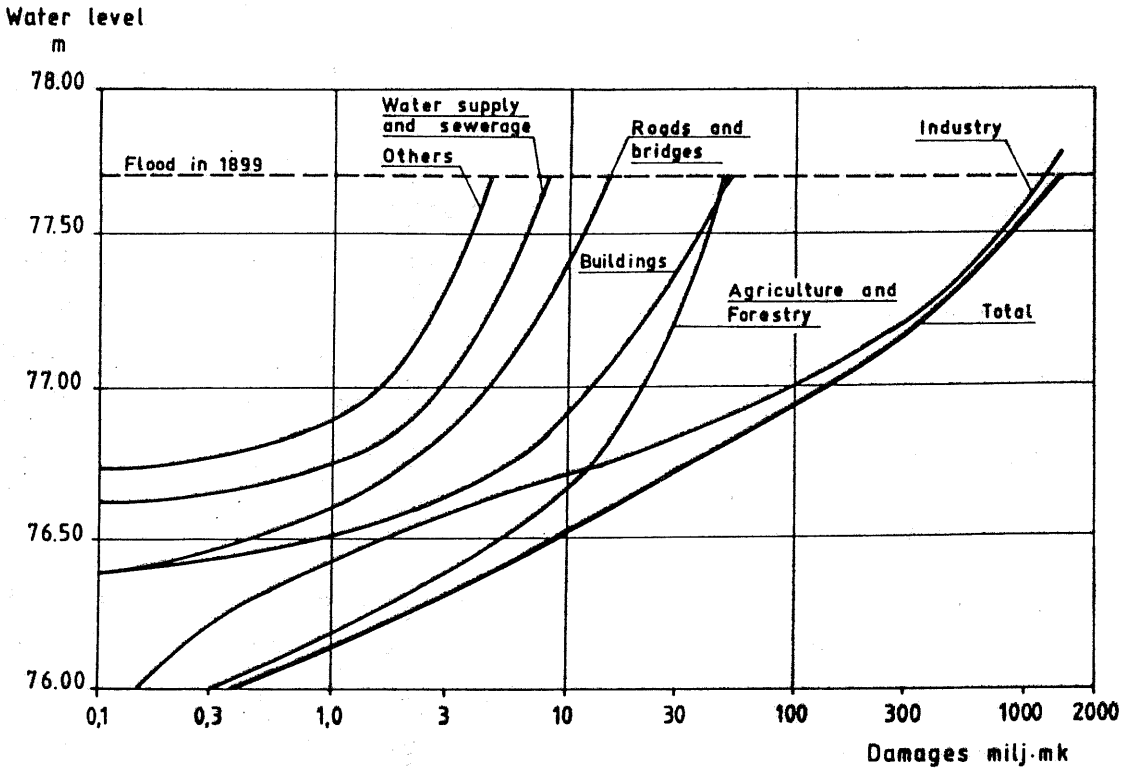


Fig. 2 Flood damages at Lake Saimaa as a function of water level.

The largest regulations in the Vuoksi watercourse are quite moderate. They all tend to lower the highest flood levels and raise the lowest water levels. Flow regulation provides means for preventing damages caused by floods and droughts. When there is a threat of very great damage, a temporary permit to deviate from normal permit conditions can be applied for. These exceptional permits have quite often been used in recent years for lakes Saimaa and Pielinen (Table 1).

Table 1. THE EFFECT OF EXCEPTIONAL FLOOD-REGULATIONS EXECUTED IN THE VUOKSI WATERCOURSE

	MAXIMUM WATER LEVEL (NN+M)			MAXIMUM DISCHARGE (M^3/S)		
	NATURAL	REGULATED	DIFF.	NATURAL	REGULATED	DIFF.
LAKE SAIMAA						
1962-3	76,79	76,74	-0,05	896	1 109	+213
1974-5	76,90	76,78	-0,12	932	1 115	+183
1981	76,85	76,61	-0,24	916	923	+7
1982	76,87	76,51	-0,36	922	1 061	+139
1984	76,62	76,31	-0,31	841	908	+67
LAKE PIELINEN						
1981	94,88	94,59	-0,29	546	584	+38
1982	94,52	94,35	-0,17	446	448	+2
1984	94,48	94,06	-0,42	435	438	+3

Also different flood control structures, such as dredgings and levees, have been built in the Vuoksi drainage basin. On the shores of lake Saimaa, about 50 separate embankments have been implemented. With these, 3000 hectars of shoreland have been protected against floods.

The operational management includes the inflow forecasts, which are made monthly for the central lakes. The independent variables are the average inflow of the previous month and, in some models, the precipitation of previous month as well. The correlation coefficients of inflow models for Lake Saimaa are 0.83...0,96. In addition to the average forecasts, maximum and minimum forecasts using 10 % probability values are calculated as well.

A convention concerning the transboundary waters between the USSR and Finland was concluded in 1964. According to this convention information concerning the discharges of River

Vuoksi is sent weekly in advance to the hydropower station at Svetogorsk. The forecast for the following two months is sent to the power station, as well.

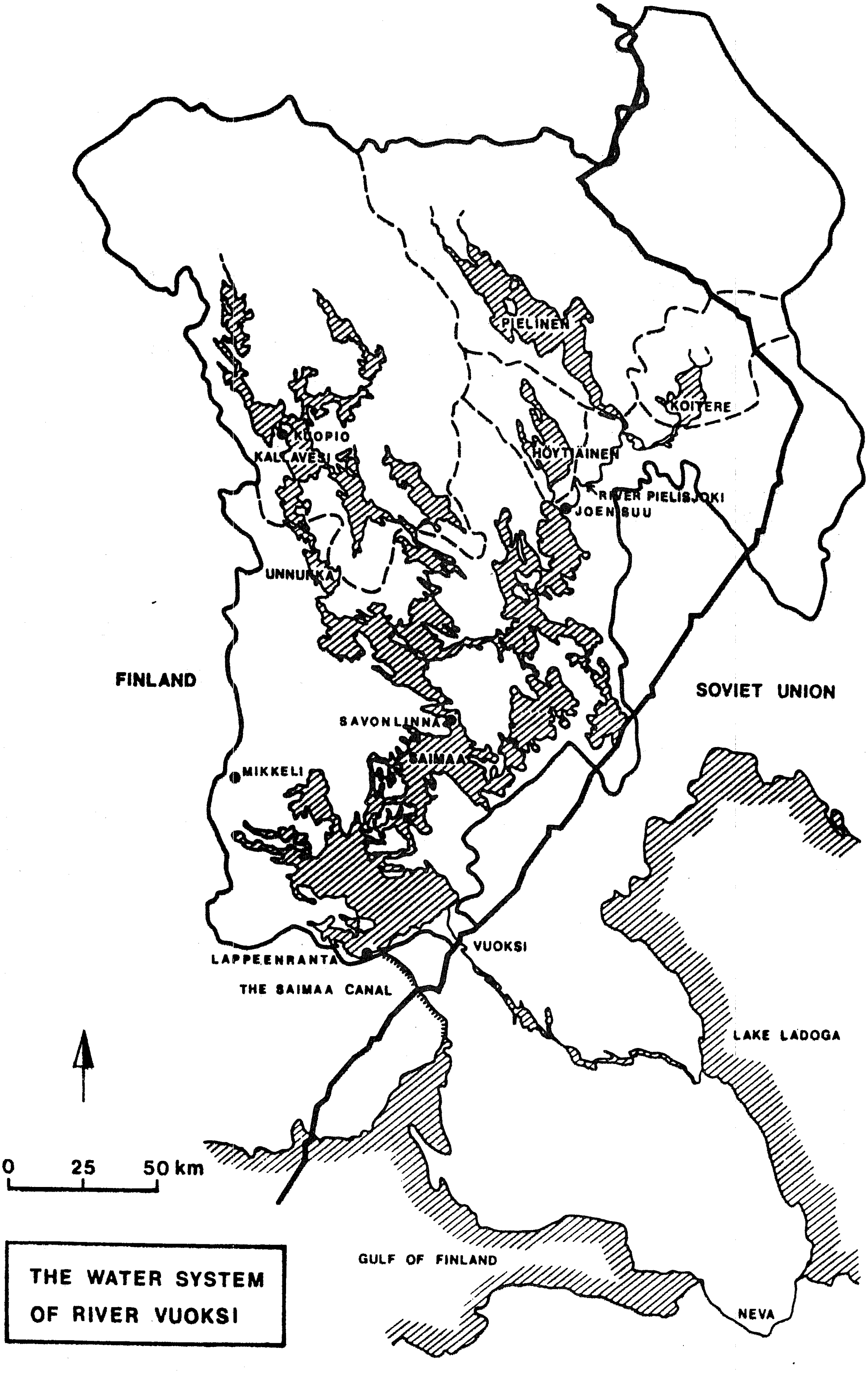
According to the convention concerning the transboundary waters, negotiations are held in the commission for the planning and execution of exceptional flood regulation of lake Saimaa. In these negotiations, discharges are settled as well as the time for the next contact to evaluate the situation. In practice, co-operation has been quite flexible.

Exceptional discharges have significantly reduced flood damages. On the other hand, these discharges have caused energy losses to hydropower plants both in the USSR and Finland. The energy losses caused by exceptional discharges of Vuoksi have been compensated to the Soviet Union.

The planning work to control the floods of lakes Saimaa and Pielinen has been going on for a very long time. Many plans have been completed, but none of them have been implemented. For this reason, the flood control of the lakes has been carried out through exceptional discharges. The problem is that, according to the Water Act, a permit may be obtained only if it can be indicated that considerable damage is to be expected. This leads to the exceptional discharges being started too late and, therefore, the reduction of flood water level is not sufficient. The possibility to use exceptional discharges has been impaired by the new investigations which indicate that the flood damages along river Vuoksi in the USSR are very great.

The action plan for flood control of the Vuoksi watercourse is under preparation. In the plan, separate alternatives will be presented for reducing flood damages. The aim of the plan is to develop arrangements and procedures by which the benefits of both countries can be equally taken into account while reducing the damages caused by floods.

APPENDIX 1



THE WATER SYSTEM OF RIVER VUOKSI

APPENDIX 2

HYDROLOGICAL DATA OF THE LARGEST LAKES IN THE VUOKSI WATERCOURSE AND OF LAKE LADOGA

	MEAN WATER LEVEL	SURFACE AREA AT MEAN LEVEL	MEAN DEPTH	VOLUME BETWEEN MEAN AND MINIMUM LEVELS	CURRENT WATER LEVEL VARIATION (CM)		DISCHARGE IN 1961-80 (M^3/S)		
	(NN+M)	(KM^2)	(M)	($10^6 M^3$)	ANNUAL MEAN	MAXIMUM RANGE	MQ	HQ	NQ
SAIMAA	75,63	4 368	14	3 672	68	161	554	1 115	165
PIELINEN	93,51	960	9	665	119	168	231	545	45
KALLA-VESI	81,54	898	9	561	71	160	166	484	52
UNNUKKA	81,07	110	-	51	28	60	118	358	30
HÖYTI-ÄINEN	87,24	290	8	169	78	85	14	72	0
KOITERE	143,02	181	6	173	154	215	73	263	1
LADOGA[1]	4,8	17 840	51	14 986[2]	74[2]	164[2]	2 540	45	10 687

[1] DATA FROM 1859 - 1956

[2] ESTIMATED ON THE BASIS OF L. SAIMAA VALUES

AN ANALYSIS OF COMPLEX WATER RESOURCES SYSTEMS PERFORMANCE. PROBLEMS AND METHODS OF RESEARCH.

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Scientific studies, carried out in the frames of the problem of rational use and preservation of water resources in the USSR, are based on the theory of runoff control developed by Soviet scientists S.N.Kritskii and M.F.Menkel in the 1930-s [1]. The theory has worked out a number of hypotheses, statements, assumptions and constraints, facilitating the analysis and estimation of probabilistic parameters of the river runoff as a stochastic process and providing a possibility of deterministic decision making, concerning parameters and regimes of hydrotechnical projects' performance. The viability of this theory is corroborated by the practice of construction and operation of many multiple-purpose hydrotechnical projects in the USSR and abroad.

It is known, that, as a rule, water consumption by various branches of national economy presupposes temporal and spatial regulation of water resources. Degrees and schemes of control are differentiated. Usually, the main objective is to provide some water- or energy yield to meet the demands of the priority water users (water supply, industry, large hydroelectric plants, water transport) and, at the same time, to use the remaining part of water to meet the demands of secondary users as much as possible (fishery releases, agriculture).

To reach this objective to the highest extent is possible only when each reservoir functions by the rules, determining the order of water accumulation and discharges, dependent on hydrologic state and water users' demands. The gained experience indicates that the lack or violation of these rules results in considerable damages [2].

In the USSR and abroad the most widely practised operating policies are based on the principle of "dispatcherization" or "dispatcher rules", which allows us under streamflow and water demands stochasticity to make deterministic solutions, concerning releases from reservoirs (volumes and hydrographs, an admissible level of fluctuations in reservoirs and maintenance of some definite levels in a river).

If reservoirs serve multiple purposes (water- or energy supply, flood protection,etc.) trade-offs among alternative objectives of runoff regulation are aggravated, thus complicating the operating rules. The use of dispatcher rules ensures realization of the major part of the theoretically possible effect from runoff regulation.

Further improvement of the operating policies should be based on the use of hydrologic forecasts. The probabilistic sense of forecasts consists in the transition from unconditional distribution of values of the future runoff to conditional distribution, taking into account the dependence of the future runoff on the factors already manifested at the time of working out the forecast. Seemingly, the modern level of anticipation (earliness of forecast), provided by hydrologic forecasts, could not influence the guaranteed regime of water supply. However, they may be useful in periods preceeding low-flow periods, exceeding the estimated reliability, or in flood periods.

Two approaches are usually used to identify optimal parameters of hydroprojects and rational rules of their operation, namely: 1) the regime of runoff regulation is calculated for the historical observed runoff series; the results of the calculation are used for the assessement of probability distribution of water resources characteristics; 2) the regime of runoff regulation is calculated by analysing probabilities of different alternations of dry and wet years and seasons by composition or statistical tests.I2 I.

Nowdays the first approach to water resources calculations is widely used. It is, undoubtly, a probabilistic means of computation - it uses a period of observations as a statistical sample and projects stochastic regularities of runoff fluctuations, observed in the past, for the future.

This method provides satisfactory results for extended periods of observations and low degree of the multiyear or seasonal runoff control.

All these general statements and practical means of the theory of runoff regulation have been developed in detail for a single- or two purposes reservoir. Specific demands for initial hydrologic information have been identified. The relation "reservoir storage - its yield - reliability of water supply" has been substantiated.

At the same time, the modern stage of water resources development in the USSR and abroad is characterized by the development of complex water resources systems, incorporating a number of reservoirs on one and the same or a number of rivers. Such water resources systems are usually hydraulically interconnected, besides that, these systems are complex, as they serve to meet the demands of various national economic branches. As a rule, the demands of separate branches on water resources systems are controversial.

Another peculiarity of water resources systems is their close interrelation with the environment as through water resources, considerable regulation of which could impose major changes on the water regime of ecological systems, and also through water users, directly withdrawing water from a water source and polluting the environment by return and sewage waters.

All these factors complicate water resources systems. The selection of parameters of the water resources systems and operating policies is one of the most difficult problems, involved in rational use and preservation of water resources.

The problem is multiaspect:

a) the development of approaches and techniques for preparation or forming of initial hydrologic information to determine conditions of systems' functioning in the future;

b) identification of demands of various branches of national economy on the regime of the system's water resources use;

c) the development of methods for the solution of the problem of the supply/demand relations, i.e. the problem of identification of parameters and operating rules for complex water resources systems under their integration.

The first two aspects form the original fields of studies. The specific feature of these studies on the modern stage is the necessity to take into account the increasing antropogenic influence on water resources formation on the water catchment, on the one hand, and the growing attention to ecological and environmental demands (e.g. to provide releases into seas to maintain the salt and level regimes, maintenance of sanitary conditions in a water route, fishery and other special releases, etc.), on the other.

All these factors necessitate the development of a new approach to the problem of identification or revision of parameters and operating policies for a complex system of reservoirs.

To solve such problems the method of analysing a complex water resources system's functioning under different hydrologic conditions and hypotheses or scenarious of national economic development is used. The initial hydrologic data is used as the observed historical or modelled hydrologic series.

As it has been mentioned, the objective of analysis of water resources systems functioning or simulation of its behavior under different conditions in the future is the selection or identification of parameters and a regime of the existing WRS (water resources systems), functioning for a long period of time.

Of utmost importance, nowdays, are the problems of revision or specification of parameters, rules, major objectives of WRS. On the one hand, the long time for planning, construction and management hampers the correct prediction of their performance in the future. On the other hand, during the last two decades our views on water value have substantially changed. If, earlier, we considered water to be mainly the energy or transport resource now the emphasis has shifted to the use of water for preservation and enhancement of ecological systems in the river and sea basins; in this regard, there is no substitute for water. For such problems the analysis of WRS performance in the past and simulation of its behavior under different states of water bearing in the future is used to assess adaptive features of the system, i.e. to evaluate its possibilities to preserve its viability, resiliency, flexibility under changing hydrologic and national economic conditions.

A key issue in the analysis of complex WRS functioning is identification of a system of indices, sufficient for determination of the WRS parameters and regime and for assessment of its adaptive features. Seemingly, first of all they should incorporate the hydrologic and water resources indices, characterizing reliability of the guaranteed yield, system's performance under strained situation with water, the degree of runoff use,etc.

Reliability is the main index, characterizing the parameters and regime of WRS, its adaptive features. In the USSR indices of reliability of the guaranteed water supply are differentiated for various branches of national economy - the number of uninterrupted years, the length of an uninterrupted period, the volume of undersupplied production, the regularity of some definite releases from reservoirs, etc. Usually, they are expressed in percentage, related to the aggregate water use under regular (uninterrupted) water supply. In the analysis of the WRS operation stage in the future reliability characterizes mathematical expectation of the corresponding values.The most comprehensive index of water supply reliability is the volume of the water supplied; the most widely used and useful - the number of uninterrupted years.

In theory the estimated reliability should be determined for each particular case on the basis of technical and economic computations. Due to the difficulty of correct assessment of damages from water undersupply (limitations) in the USSR in real practice reliability on a number of uninterrupted years is standardized:

- industrial and municipal water supply - 95-99%;
- hydroenergy production - 90%;
- agriculture - 75%.

Modern standards of the estimated reliability are not absolute, as they depend on water users only and do not account specific features of a particular water source. Studies on substantiation of the standard values are being held. Nevertheless, at the present time the standards are the major indices of the complex water resources systems functioning.

On the modern stage an analysis of WRS performance is possible only on the basis of mathematical models. Presently, a great number of mathematical models: optimization and simulation, static and dynamic, stochastic and deterministic has been developed. All of them are viable, it is expedient to select a proper mathematical model for a particular WRS and a problem, its base, the taken hypothesis, assumptions, simplifications.

In our studies we use the principle of simulation modelling and simulation models. This method allows us not to obtain initial economic data, not to substantiate the economic criterion or objective function and not to use some optimization algorithm. At the same time, simulation models allow one to obtain data on WRS performance under different conditions for further decision making on its parameters and regimes.

The above statements can be illustrated on a case study of the Volga River WRS - a problem of revision of the major WRS parameters, its regulating capacity and concretization of operating policies for the system. (Table).

Runoff of the Volga and Kama Rivers is regulated by a cascade of 11 reservoirs with total volume of 88 km^3. The first hydroprojects were constructed in the 1930-40-s and the last ones were constructed not long ago. The major water users are water supply, energy production, water transport, agriculture, fishery and recreation. As it was designed by the project, the main objective of water reservoirs was to meet the demands of energy production and to maintain navigable levels of the rivers Volga and Kama.

An analysis of experience of the cascade's functioning for the last 25 years shows that the actual economic situation and, therefore, the objectives of WRS differ from the projected ones. Without dwelling on changes in conditions of energy use, we shall ennumerate the new demands of non-energy users (spring ecological releases, considerable consumptive water withdrawals in a basin, etc.). An analysis of WRS functioning shows that energy and water transport demands, despite further complications of conditions, were provided with reliability similar to the projected one.

Priority was given to hydropower plants, used in an integrated energy system not only for generation of daily peak capacity yields, but to cover energy deficits by use of water from reservoirs in the autumn-winter period. Such regime of water resources use contradicts with the demands of other water users, as the releases from reservoirs for energy purposes before flood periods necessitate an increase in amounts of water for reservoir filling in spring by means of decrease of a spring ecological release. in the lower reaches of the Volga River.

In the lower Volga from the city Volgograd up to the estuary of the river there are trade-offs between the interests of major water users: fishery, agriculture, ecological demands of the northern Caspian Sea, energy production, navigation. Demands on the water regime of this region considerably influence the regime of work for the whole cascade. Economic development in the lower reaches of the Volga River has formed in conditions of natural floodings of the Volga-Akhtuba floodlands and its delta by spring water. Runoff accumulation in reservoirs in spring (usually in amount of 50-60 km^3) changes conditions of flood formation in the Lower Volga.

Slow implementation of measures on reconstruction of fisheries and agriculture in the lower reaches of the Volga and low efficiency result in the necessity to provide high and long water releases from the Volga reservoirs. The optimal volume of the spring releases with account of fishery and agricultural demands is approximately 120 km^3 [4]. However, to provide such a release annually would mean impossibility to accumulate the necessary volume of water to meet the guaranteed demands of other water users. Therefore,"compromising" regimes of releases amounting to 95-105 km^3 with the 70% probability (for a number of uninterrupted years) have been additionally developed.

However, the analysis of the cascade's functioning has shown that the average release during the period of 1960-1984 years was approximately 97 km^3 with the average inflow 154 km^3 ($P \sim 50\%$), i.e. fishery releases in the lower reaches of the Volga River even in a "compromising" volume were not reached [3, 4].

Therefore, fisheries in the Volga River basin in conditions of runoff regulation suffer considerable difficulties, caused by changes in natural habitat in the reservoirs and in the Lower Volga. To improve this situation under runoff regulation is possible by reduction of the volume of releases from the reservoirs in the autumn-winter period for energy production, leading to an increase in the volume of fishery releases in spring in the lower reaches of the Volga River.

Reliability of water supply of various branches of national economy is determined by an active or a regulating volume of the reservoirs of the cascade. Three alternatives of the active volume were analysed in our studies - 88, 78 and 68 km^3; the total active storage of the cascade (from normal level to dispatcher level) is to be 71, 61, 55 km^3, accordingly.

The first alternative corresponds to the use of the whole projected active storage of reservoirs of the Volga-Kama cascade after the completion of its construction (Cheboksarskii and Nizhnekamskii hydropower plants included).

The second alternative rejects the use of the Cheboksarskii and Nizhnekamskii reservoirs for runoff control.

The third alternative presupposes the decrease of total regulating storage of the cascade to 68 km^3 by means of changes in the level of water releases before a flood period from the Kuibyshev reservoir from 45.5 to 48.0 m. Computations have been carried out for the 26-year historical hydrologic series (from 1914/1915 till 1939/1940 years) on the basis of a mathematical model and a computer programme [5,4].

An analysis of the obtained results has shown that, as far as fisheries in the Lower Volga are concerned, with the reduction of total regulating volume of the cascade the probability of spring releases increases. However, even under considerable (up to 20 km^3)reduction of regulating storage of the cascade in case of the third alternative it is possible to get only a slight increase in reliability of spring releases in the Lower Volga. Reliability of a release amounting to 120 km^3 increases from 31 to 38 %, reliability of a release amounting to 100 km^3 from 40 to 53%, and reliability of a release of 70 km^3 from 70 to 80%. (Fig.1).

Reliability of normal releases for navigation downstream the city Volgograd in amount of 4000 m^3/s is approximately equal for all three alternatives of the total active volume of reservoirs of the cascade and accounts for 78%; the reduced release 3400 m^3/s has an 85-88% probability. Such a great decrease in reliability of the navigation release compared to the projected reliability (95-97%) is caused by the necessity to provide fishery releases not incorporated into the project by consumptive water withdrawals in a basin and by some hydrologic peculiarities of the computated runoff series. As for the energy effect, in case of the first alternative the increases in the mean annual energy yields for 0.6 milliard KWTh are observed (Fig.2). For this alternative a real danger arises of substantial undersupplies of transportation demands in low flow years; reliability of a spring fishery release to the lower riches of the Volga River decreases. Typical for this alternative are the extremely large winter releases of water downstream the Volgograd hydroproject, resulting in a number of cases in underflooding of the Volga-Akhtubinsk flood-lands and the delta of the Volga River in winter conditions, not desired from environmental and economic points of view.

Therefore, as for environmental demands, it is desirable to reduce the total regulating capacity of reservoirs. This can be done by rejection of the use of the Cheboksarskii and Nizhnekamskii reservoirs and increase in dispatcher levels of releases from the Kuibyshev or other reservoirs of the cascade.

From the power production point of view the more optimal is the alternative with the higher active storage of 78 km^3. Just as in the case of the Don WRS we have to tackle the same problem - the change of traditional scheme of runoff regulation to improve ecological situation always results in undersupply of such water users, for which it was primarily projected. Possibilities of the system to adapt to new conditions of water control under maintenance of the projected demands for amounts of water and reliability of its acquisition are limited. Therefore, as environmental aspects are one of the major issues in the problem of water resources use, they can be solved only by means of a system of limitations laid on water users, demanding water in low flow periods. These users should adapt to a new regime by a more rational water development and the use of more complicated operational rules.

Table

The Major Parameters of Hydroprojects of the Volga-Kama Cascade

Hydroproject	The mean annual runoff, km^3	The reservoir volume, km^3		Installed capacity, MWT	The mean annual production, million KWTh
		full	active		
Volga					
Ivankovskii	9.6	1.12	0.8	30	127
Uglicheskii	13.6	1.24	0.8	110	240
Rybinskii	35.2	25.4	16.7	330	1100
Gorkovskii	52.5	8.5	2.8	520	1513
Cheboksarskii	112.7	11.2	5.7	1404	3340
Kuibyshevskii	241.0	57.0	33.6	2300	10900
Saratovskii	247.0	12.9	1.8	1360	5400
Volgogradskii	251.0	31.4	8.2	2563	11100
Total for Volga		148.76	70.4	8617	33720
Kama					
Kamskii	51.6	12.2	9.8	504	1915
Votkinskii	53.7	9.4	4.5	1000	1915
Nizhnekamskii	89.3	13.6	4.6	1248	2540
Total for Kama		35.2	18.9	2752	6775
Total for Volga-Kama cascade		183.96	89.3	11369	40495

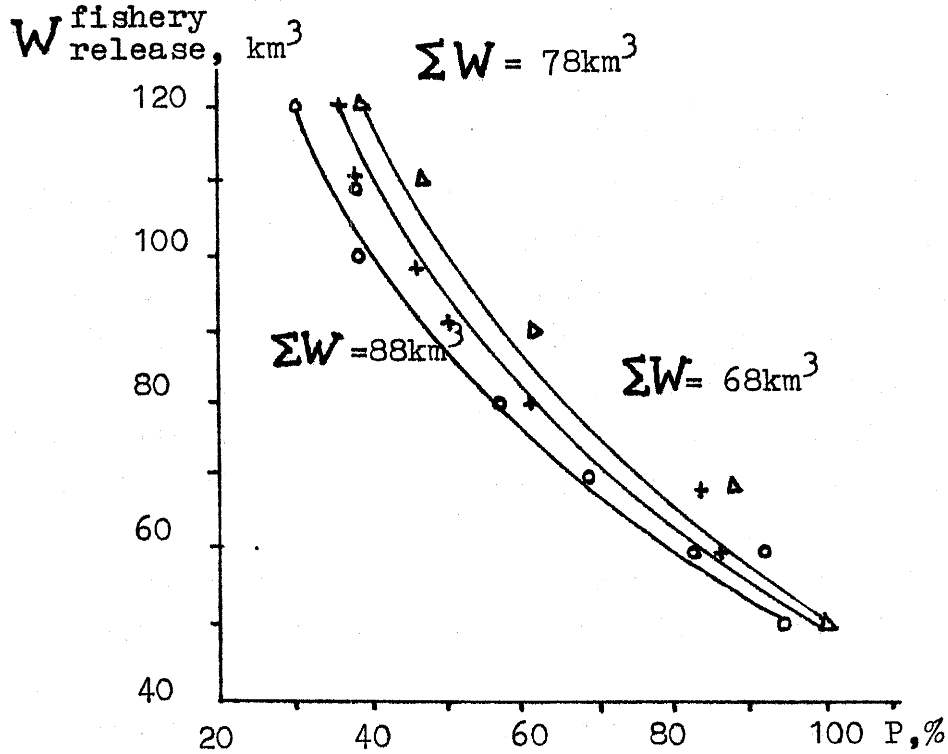

Fig.1. The curves of probability of spring releases into the lower reaches of the Volgograd hydroproject for alternatives of the regulating volume of the cascade (ΣW).

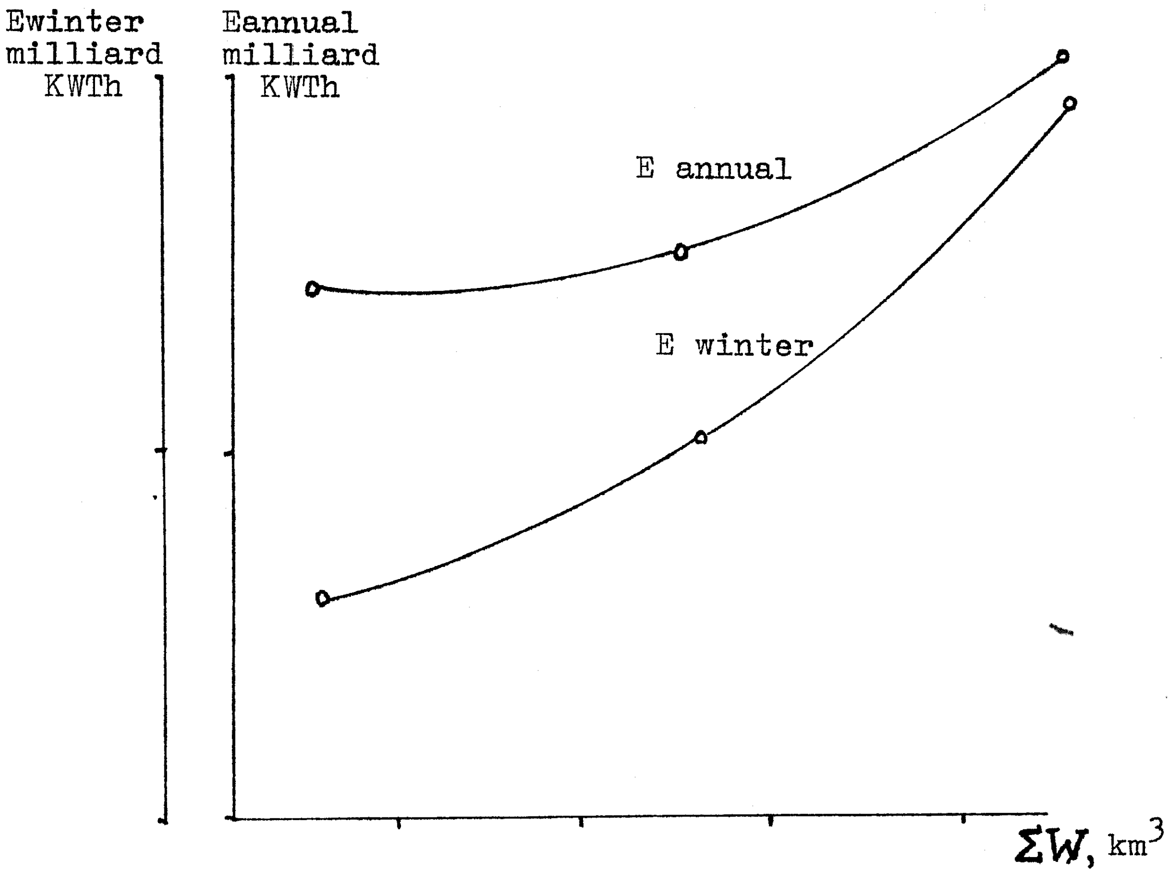

Fig.2. The dependence of hydropower production of the Volga-Kama cascade on the value of the total regulating volume(ΣW)(absolute values)

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THE POSSIBLE USE OF HEADWATER LAKES IN THE VUOKSI RIVER BASIN FOR FLOOD CONTROL OF LAKE SAIMAA AND DURING DROUGHTS

M. Maunula

1. INTRODUCTION

One of the subjects of the co-operation program between the Finnish National Board of Waters and Environment and the Water Problems Institute of the USSR Academy of Sciences is the development of operating rules for a water resource system which includes lakes and reservoirs, with account to diversion of river flow. The main purpose is to apply the mathematical model developed and used in the Water Problems Institute to the design of the operational rules for the Vuoksi Water System in Finland. The work is aimed at the development of methodology and policy for operation of multiple-use water resource systems in conditions of uncertainty, especially for flood and drought periods.

In this presentation, the possibility to use the upper lakes of the Vuoksi Water System in the flood control of lake Saimaa and during exceptional droughts has been considered. In the paper by Porttikivi and Maunula, the hydrology of the Vuoksi Water System and management and flood control of the Vuoksi river system has been presented.

A major difficulty in the utilization of the Vuoksi Water System is, on one hand, the forecasting of exceptional flood and drought periods and, on the other, the compiling of operation rules in such a way that overall damages are minimized. During flooding, the main aim is to lower the water stage of lake Saimaa and to avoid excessive discharges into the Vuoksi river.

In a situation where the "exceptional discharges" are not sufficient, one has to consider the use of upper lakes for minimizing the overall damages. In order to find the right decisions, the hydrological dependencies of lakes of the Vuoksi Water System have to be investigated.

2. MATERIALS AND METHODS

In the consideration of the hydrology of the lakes of the Vuoksi River Basin, the multireservoir regulation model developed in the USSR has been used. The model optimizes the use of the storage of the lakes and discharges. The input data needed are the monthly inflows, regulation rules of the lakes and priorities of filling the lakes. The model has been described by Korobova, Oziranskiy, Poizner: "Analysis of the water management system of the Vuoksi river basin functioning as the basis for the control rules compilation".

The regulation model includes lakes Saimaa (4 500 km^2), Kallavesi (900 km^2), Unnukka (110 km^2), Pielinen (960 km^2), Höytiäinen (290 km^2) and Koitere (180 km^2) (appendix 1).

The time series simulated by the model are the years 1941 - 1954 and 1956 - 1982. Year 1955 is not included because of the lack of discharge observations at lake Koitere. For the simulation, monthly targets of water surface elevations were prepared also for lakes Pielinen and Saimaa although those lakes are not regulated at present. The aim of this simulation was to discover the effect of the regulation onto the joint behaviour of the lakes. Computational time step is one month.

The net inflows of the lakes have usually been calculated by the storage equation using observed water levels and discharges. The monthly net inflows into lake Saimaa from the immediate drainage area have been calculated so that from the total inflow values of the Vuoksi Water System the observed discharges of headwater-lakes have been subtracted. The inflows to lake Unnukka have been calculated on the basis of the inflows to lake Kallavesi.

At the same time as the calculation of inflows was worked out, the developping of forecasting equations was started. The inflow models were made by stepwise regression. The inflow models for spring floods were made for all lakes and the monthly inflow models from July to February for lakes Saimaa, Kallavesi and Pielinen. The difference between the observed inflows and the predicted inflows was also investigated, and the corresponding recurrences were calculated for these differences. These analyses have been investigated more closely in a presentation by Poizner and Plotkin: "Use of long-range forecasts of unregulated inflow to Vuoksi river basin lakes in problems of lakes levels control".

3. RESULTS

The central lakes of the Vuoksi watershed are, as for the Finnish conditions, rather large. Nevertheless, the operative possibilities for the lakes above Saimaa are limited, because the operational storage capacity of these lakes is only about one half of the storage capacity of Lake Saimaa. Of main importance for the operative use of the waters are lakes Saimaa, Pielinen and Kallavesi (table 1).

Table 1. The storage capacity of upper lakes in relation to lake Saimaa

	SURFACE AREA	RATIO TO THE SAIMAA SURFACE AREA	USE OF STORAGE CAPACITY DURING 6 MONTHS	THE CORRES-PONDING CHANGE IN THE WATER LEVEL OF UPPER LAKES	EFFECT ON THE SURFACE ELEVATION OF LAKE SAIMAA
	(KM2)		(M^3/S)	(CM)	(CM)
PIELINEN	960	1 : 4,6	62	100	22
KALLAVESI	898	1 : 4,9	29	50	10
UNNUKKA	110	1 : 40	1	20	n. 0,5
HÖYTIÄINEN	290	1 : 15	4	20	1
KOITERE	181	1 : 24	6	50	2

Exceptional floods are normally multiannual, which has to be taken into account when examining the operational possibilities of the headwater lakes. Because of the limited storage capacity of the headwater lakes, it is not practicable to operate them for the flood control of lake Saimaa in the first year. If an extraordinary flood is threatening, first the flood discharge of lake Saimaa has to be increased as early as possible. In the multireservoir simulation the discharge of river Vuoksi is increased to the rated discharge value of 850 m^3/s. A successful discharge forecast of two months means a 10-15 cm decrease in flood level.

For example as a result of the model simulation demonstrates that, for the flood of 1974 - 75, the drop in flood stages in the end of 1974 resulted mainly from the increased discharge of Vuoksi, but the drop in the flood stages in the spring of 1975 resulted only from the regulation of the headwater lakes. The discharge into river Vuoksi was not higher than 853 m^3/s during the whole flood period of 1974-75 (table 2).

In the sumulation during the period from 1 Aug. 1974 to 30 Jun. 1975 (11 months), a mean value of 27 m^3/s was stored in Lake Pielinen and 19 m^3/s in Lake Kallavesi. The benefit was a decrease in the flood stages of lake Saimaa by approx. 20 cm to NN+76,66 m (monthly average value). The stage of lake Kallavesi rose to NN+82,85 m and that of lake Pielinen to NN+94,70 m. According to the preliminary damage analysis, Kallavesi rose about 30 cm too much and, correspondingly, more could have been stored in lake Pielinen (table 2, appendixes 2-4).

Table 2. Maximum water surface elevations and discharges of Lakes Saimaa, Pielinen and Kallavesi, simulated results compared with natural and observed monthly average values.

Saimaa

	Maximum waterlevel NN+m			Maximum discharge m^3/s		
year	natural	regulated	diff.	natural	regulated	diff.
1943	76,29	75,99	-0,30	737	838	+101
1954	76,07	76,25	-0,18	671	849	+178
1962	76,76	76,19	-0,57	886	849	- 37
1963	76,72	76,00	-0,72	873	849	- 24
1974	76,73	76,52	-0,21	875	849	- 26
1975	76,88	76,66	-0,22	928	853	- 75
1981	76,82	76,45	-0,37	905	849	- 56
1982	76,83	76,36	-0,47	909	849	- 60

Pielinen

year	observed	regulated	diff.	observed	regulated	diff.
1943	94,69	93,82	-0,87	322	241	- 81
1954	94,26	93,82	-0,44	265	279	+ 14
1962	94,64	94,84	+0,20	403	279	-124
1963	94,06	94,71	+0,65	213	227	+ 14
1974	94,35	94,10	-0,25	300	279	- 21
1975	94,38	94,70	+0,32	282	239	- 43
1981	94,56	94,57	+0,01	341	279	- 62
1982	94,30	94,92	+0,62	284	279	- 5

Kallavesi

year	observed	regulated	diff.	observed	regulated	diff.
1943	82,48	81,88	-0,60	467	395	- 72
1954	82,10	81,88	-0,22	311	429	+118
1962	82,24	81,88	+0,36	360	345	- 15
1963	81,68	81,80	+0,12	184	205	+ 21
1974	82,12	82,37	-0,25	376	489	+113
1975	82,21	82,85	+0,64	365	300	- 65
1981	82,43	82,47	+0,04	420	349	- 71
1982	82,31	81,94	-0,37	374	300	- 74

The drought periods of lake Saimaa are also multiannual by nature. Normally they last 0,5 - 1 year when the water level is below NN+75,25 m. An especially long drought was in years 1939 - 1943: it lasted 3 years and 4 months. In such a situation, the waterlevels of the upper lakes go down and in concequence it is not possible to increase the inflow to Lake Saimaa to a considerable extent.

During drought periods the best simulated results are achieved by lowering, in time, the discharge of Vuoksi to the value of 300 m^3/s. Normally this minimum discharge is sufficient, but during years 1939-1943 the discharge should have been decreased to 200-250 m^3/s, if the aim was to keep the water level of lake Saimaa above NN+75,10 m.

Table 3. Characteristic data of water surface elevations and discharges of Lakes Saimaa, Pielinen and Kallavesi for the time periods 1941 - 1954 and 1956 - 1982, simulated observed results compared with monthly average values

SAIMAA						
	Mean high monthly values			Mean low monthly values		
	observed	regulated	diff.	observed	regulated	diff.
NN+m	75,96	75,87	-0,09	75,29	75,46	+0,17
m^3/s	678	797	+119	429	376	-53

PIELINEN						
	observed	regulated	diff.	observed	regulated	diff.
NN+m	94,01	94,02	+0,02	92,94	92,90	-0,04
m^3/s	228	233	+5	95	82	-13

KALLAVESI						
	observed	regulated	diff.	observed	regulated	diff.
NN+m	81,99	81,94	-0,07	81,19	81,03	-0,16
m^3/s	296	277	-19	91	84	-7

4. CONCLUSIONS

The multireservoir regulation model of the watershed of river Vuoksi is used to investigate the mutual hydrological dependence of the lakes in the watershed as well as the effect of different operational alternatives. The model enables the preparation of a basic hydrological study for the Action Plan for Flood Control in the Vuoksi Water System.

According to the results, the headwater lakes can be used mainly for the flood control of lake Saimaa, not so much during droughts. Because of the limited storage volumes compared with that of lake Saimaa, these headwater lakes should be used as a last resource.

The multireservoir model presented here, has not yet been applied for proper operative use. The taking into account of different hydrological periods makes it necessary to base the operational aims on hydrological forecasts. The timestep used in the model computations is decreased to improve the accuracy of the water balance computations.

To estimate the operative possibilities of the lakes also the boundary conditions for the regulation, the flood damage at the different lakes at different water level elevations and the juridical considerations have to be taken into account. This basic information is needed for decision making in case of an extraordinary hydrological event.

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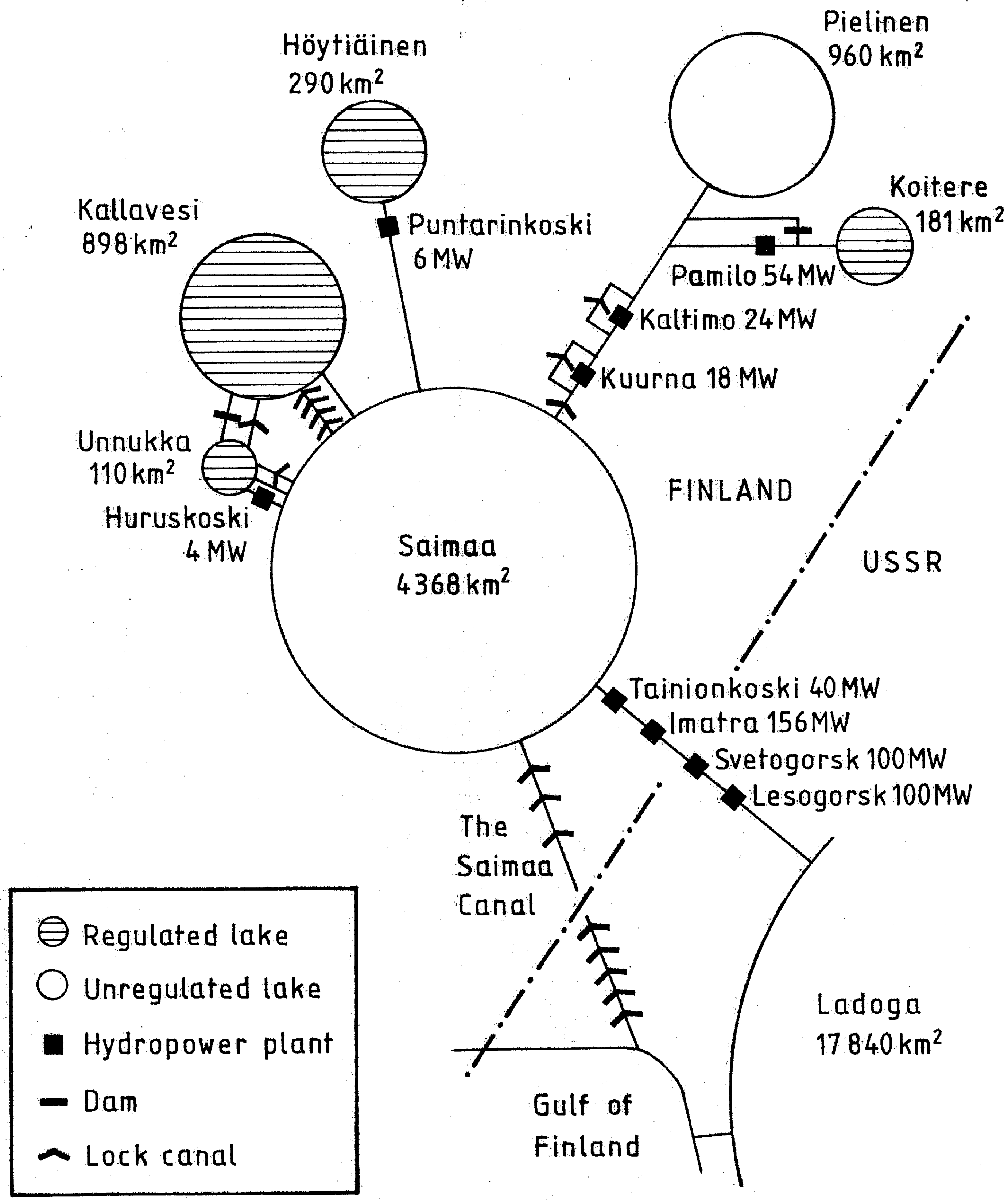
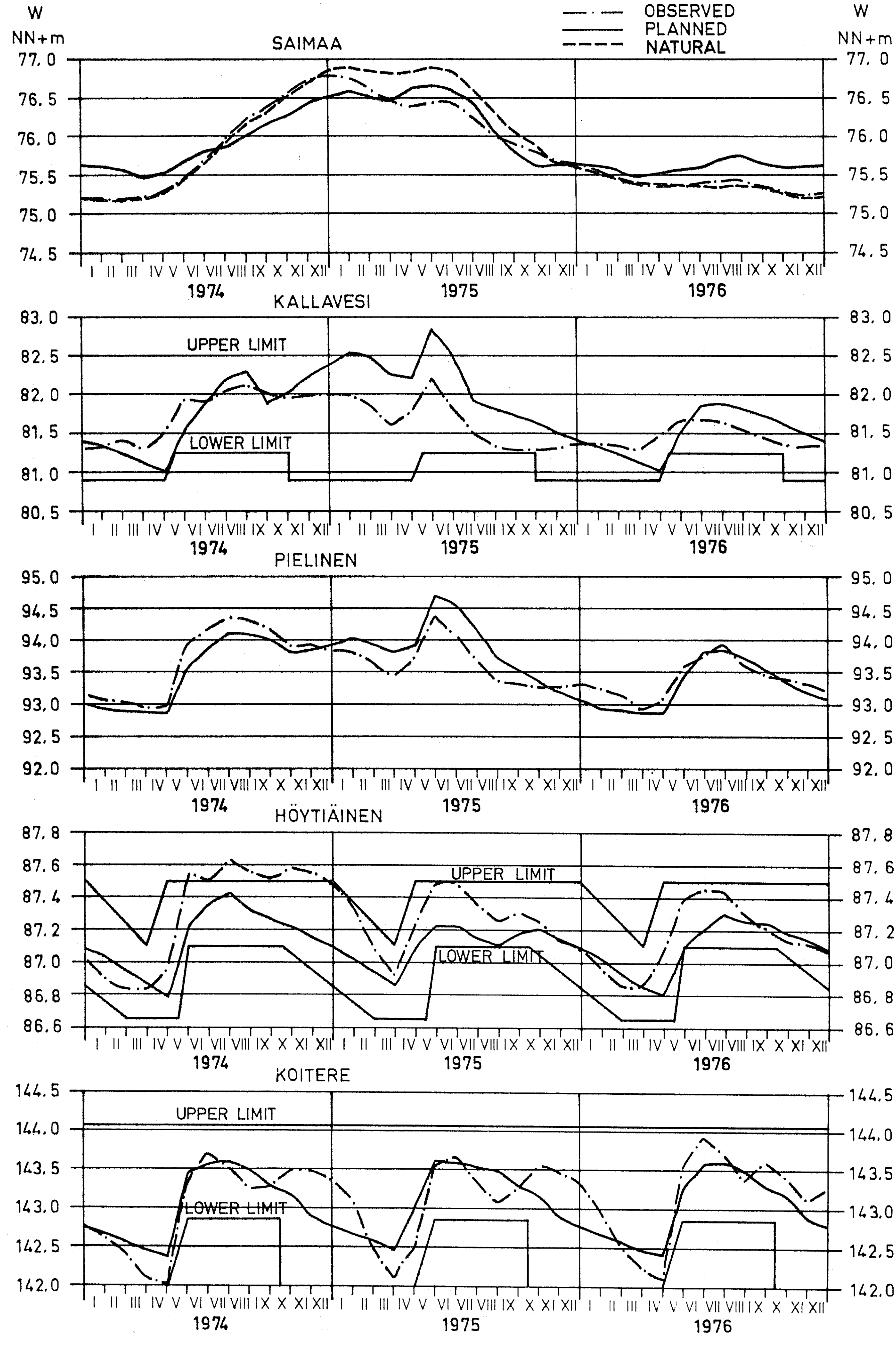
Poizner V., Plotkin Yu., Use of long-range forecasts of unregulated inflow to Vuoksi river basin lakes in problems of lakes levels control.



Shematic presentation of the Vuoksi watercourse with main hydraulic structures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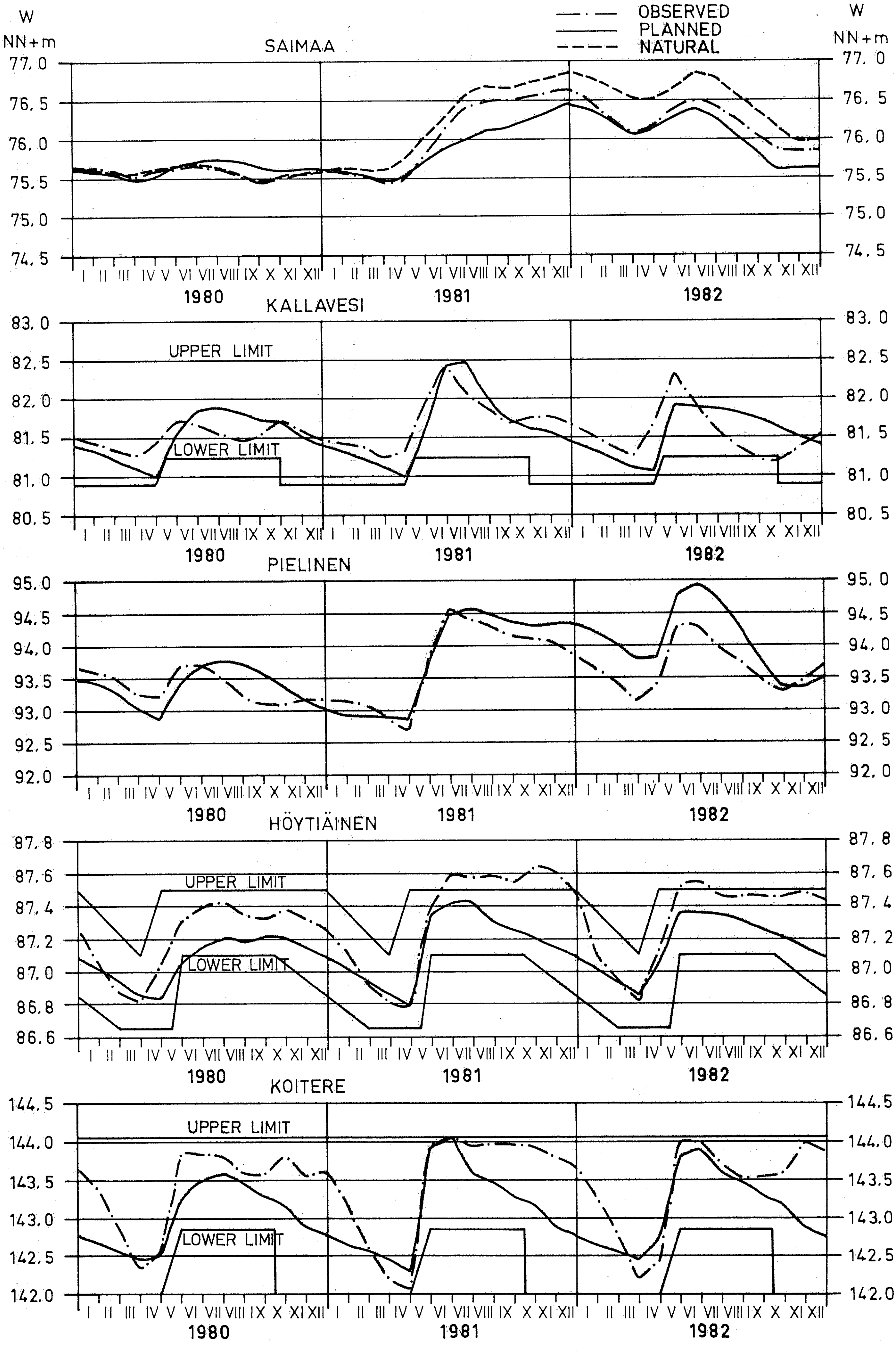
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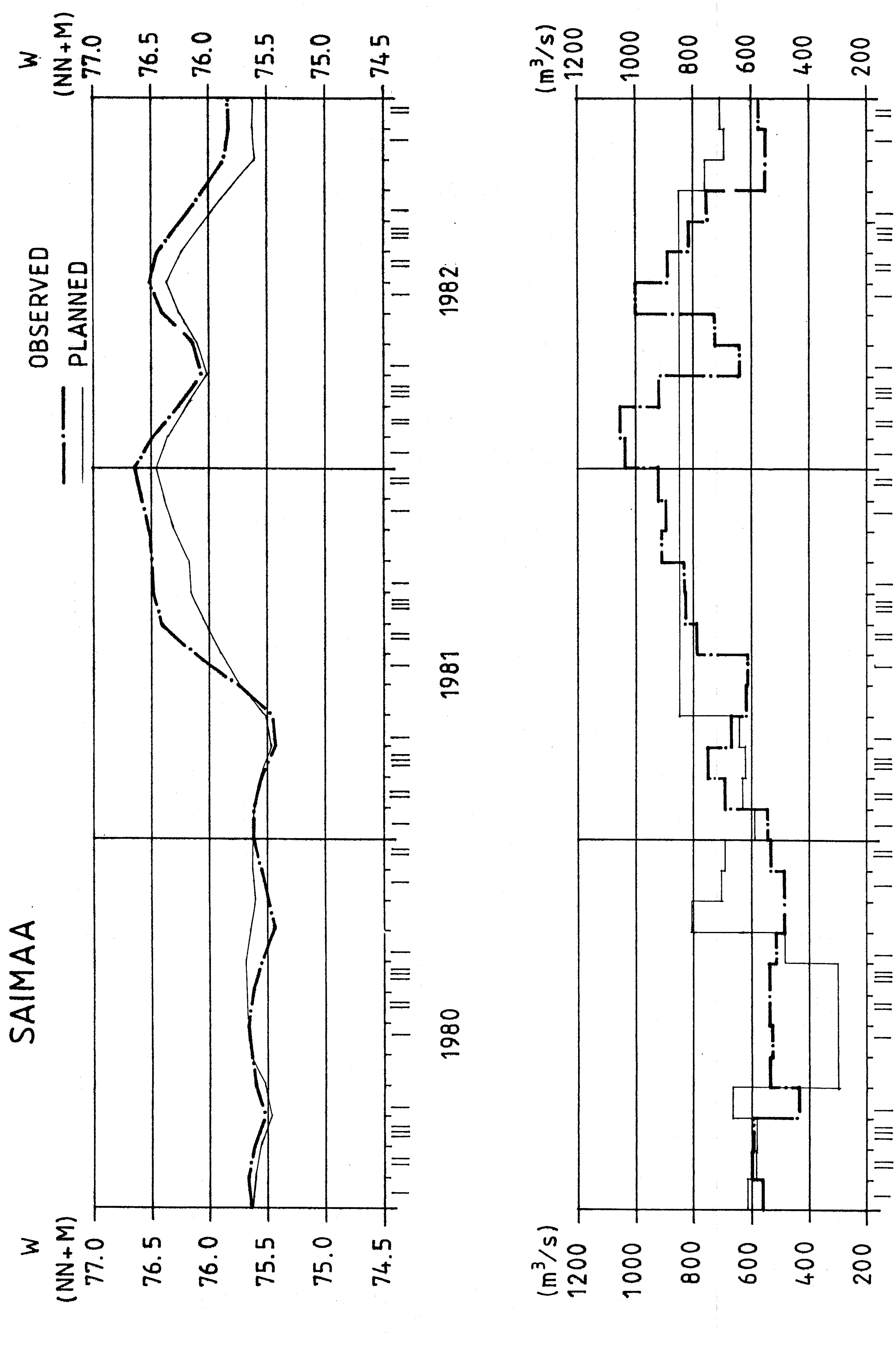
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THE DEVELOPMENT OF OPERATING RULES OF THE WATER RESOURCES SYSTEM IN THE VUOKSI RIVER BASIN BASED ON THE USE OF THE SIMULATION MATHEMATICAL MODEL

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The Vuoksi River Basin. Problems Involved in Regulation.

The Vuoksi river basin incorporates large and small lakes interconnected by short arms, forming the extensive systems of lakes and rivers. The great number of lakes in the Vuoksi river basin, the length of which is relatively short (156 km), forms a large water catchement (66160 km^2). The complexity of a hydrographic network in the basin makes direct determination of the inflow volume to the lakes impossible. Therefore, initial data on inflows are determined as the difference between the reservoir filling at the beginning and at the end of the month minus the value of visible evaporation. The series of side inflows,regenerated by Finnish researchers for the six largest lakes of the basin (Saimaa, Kallavesi, Pielinen, Koitere, Unnuka, Höytiäinen) for 1941-1954 and 1956-1982 years, incorporate negative values of inflow, indicating significant errors in determination of components of a calculated equation.

Nevertheless, as a whole, the data correspond with the seasonal variability of hydrometeorological processes. Therefore, after introduction of some corrections, caused by the necessity to eliminate negative inflow values, contradicting with its physical nature, the data have been taken as initial in computations of the given study.

The inflow to the Vuoksi river basin is unevenly distributed among lakes, predetermining their role in runoff formation of the Vuoksi river(Table 1).

Table 1.

Inflow Distribution Among Lakes in the Vuoksi River Basin

Lake \ Period	Year		Flood	
	million m^3	%	million m^3	%
Saimaa	6314	36	2903	34
Kallavesi	5022	28	2502	29
Pielinen	4901	28	2417	28
Koitere	798	4	412	5
Höytiäinen	476	3	265	3
Unnuka	211	1	105	1
Total	17722	100	8604	100

The spring flood, occuring in April-July, is an important phase of the hydrologic regime. About 40-60% of the annual runoff fall on this period, resulting in the growth of level fluctuations in reservoirs,which might damage national economic branches. Regulation of separate lakes carried out in the latest years turned to be insufficient for maintenance of an acceptable level regime of water reservoirs with the amplitude of level fluctuations limited to 10-90% probability of exceeding, going out of the limits only in extreme hydrometeorological conditions. The main attention is paid to the Saimaa lake; the increase of its level over 76.60 m leads to flooding of adjoining territories, causing damages to industrial enterprizes, recreation facilities, agriculture, and forestry. The drop of water level lower 75.10 m is also unfavourable, impeding water-born transportation, water supply, and timber floating. The necessity to provide a release not more than 850 m^3/s to the Vuoksi river limits possibilities of the lake Saimaa regulation. Exceedance of the pointed limits results in spills of the hydroelectric power station and floods on the territory of the USSR. To solve the problems it is suggested to use the developed at the Institute for Water Problems of the USSR Academy of Sciences mathematical model, based on the "out-of-kilter" algorithm.

Model's Description.

A water resources system of an arbitrary configuration is described by an oriented graph $G(IN, A)$,where $IN=\{n_i\}$ is the set of its nodes incorporating reservoirs, water users, points where inflows fall into the main water routes, intersection of channels, etc., and $A=\{(n_i,n_j)\}$ is the set of arcs connecting a pair of apexes n_i and n_j belonging to the set IN. If the arc $a_{ij}=(n_i,n_j)$ between the nodes n_i and n_j exist, water flow can be transported from the node n_i to the node n_j . The existance of such communication means the possibility of a one-way movement only (the oriented graph). In cases, where the two-way flow between the nodes is admissable, the latter should be connected by a pair of communications in the opposite directions.

For a water resources system, represented as a linear-nodal scheme, it is necessary to provide "preservation of inseparability of flows" in the apexes of the oriented graph.

The performed formalization of the water resources system allows one to formulate the problem of water resources distribution among water users on the basis of runoff regulation by reservoirs (i.e. the problem of water resources systems management in a time interval t) as a mathematical problem of cost minimization of water movement in a network.

The model can be presented in the following way - at each interval of modelling on the network $G(IN, A)$ it is necessary to find water flow, satisfying the following conditions:

$$\sum_{i\in IN} X_{ij} - \sum_{i\in} X_{ji} = 0, \tag{1}$$

$$0 \le L_{ij} \le X_{ij} \le U_{ij} \tag{2}$$

and minimizing the objective function Z

$$Z = \sum_{a_{ij}\in A} c_{ij} X_{ij} \longrightarrow min \tag{3}$$

where X_{ij} the flow in the arc $a_{ij} = \{n_i, n_j\}, a_{ij} \in A$, $n_i, n_j \in IN$ and U_{ij} is the upper limit and L_{ij} - the lower limit on the flow in the given interval of modelling. C_{ij} is the cost of transportation of a flow unit X_{ij} .

To ensure the conformity of this problem with the initial problem of optimal water resources distribution the following definitions are introduced:

1. The cost of transportation C_{ij} of a water unit in the network $G(IN,A)$ is an integral number, either positive, or negative. For such water resources systems elements as a water user or a water reservoir the cost C_{ij} is presented by a negative number, simply depending on the priority. The priority is assigned by a researcher, depending on his notions on water resources systems functioning, i.e. on the order of water supply to various water users, rules of reservoirs draw-down and their filling. Then the following condition is introduced: the higher the priority of any water resources systems element is, the less the cost of transportation of a flow unit to this element should be.

Besides that, in the model positive costs are assigned to transportation of a water flow unit through links and "system's losses", which may result from the system's functioning in unbalanced conditions.

In this case the minimal cost flow corresponds with the optimal water allocation in water resources systems, judging by the chosen system of pririties, represented in a network $G(IN,A)$

for a time interval t .

As it was already noted, priorities are assigned to water users and to desirable water volumes of a reservoir. Generally, the desirable water stores may correspond with characteristic lines of dispatcher rules, or in case of lakes - to different equally guaranteed levels. It is optional that these volumes could change, depending on the amount of water in a system or the so-called "hydrologic state of a system .

2. In the described model the hydrologic state of a water resources system for a current time interval is a certain total water volume in the system, defined as the sum of water volumes in all or some of the reservoirs at the beginning of time interval t and of inflows to these reservoirs during this interval. A possibility to introduce three hydrologic states "wet", "mean" and "dry" in the model to characterize the surplus, normal and deficit state of water in a system is envisaged. A system of priorities, definite water user's demands, and desirable water volumes correspond to each hydrologic state. To determine the hydrologic state the store of water in a system R is compared with the maximal possible store W, defined as a sum of active volumes of reservoirs. Then the dry state satisfyes the condition $R \leq \alpha_1 W$, the mean state - the condition $\alpha_1 W \leq R \leq \alpha_2 W$, the wet state - the condition $R > \alpha_2 W$.

Coefficients α_1 and α_2 , determining the bounds of hydrologic state are assigned by a researcher. To a great extent they determine the results of a simulation experiment.

3. The rules of water resources systems operation in the time interval t are called the assembly of operating parameters, determining water distribution in the system. For the given demands of water users, coefficients α_1 and α_2 should be considered to be operating parameters, determining the bounds of hydrologic state,

a system of priorities and lines of desirable water stores in reservoirs. All these parameters are set by a researcher. Identification of these parameters in a general case is relatively complex. As a rule, the parameters are chosen and corrected on the basis of analysis of such water resources indices of the system's functioning as reliability of water supply to water users, depth of interruptions of water supply, etc.

In case, when the objective of the water resources management is maintenance of the level regime in lakes, the operating parameters are corrected on the basis of the analysis and comparison of the assigned and obtained as a result of simulation experiments for a multiyear period characteristic levels.

The above mentioned definitions allow one to formulate an initial water resources problem as follows: if the chosen law of water resources systems management correspond with the researcher's ideas on water resources systems functioning peculiarities, the solution of an optimization problem for each time interval t results in identification of the amount of water supply to water users, of releases from reservoirs, and of level fluctuations in reservoirs or lakes for a number of years.

Model's verification and estimation of possibilities of its use.

The use of the described above mathematical model to develop operating rules for water resources systems in the Vuoksi river basin has demanded the development of linear-nodal schemes of the water resources system (Fig. 1a,b) - a complete and a simplifyed ones. The complete scheme incorporates six lakes (nodes 1-6), the confluence node (node 8), and the node locking the scheme (node 7 - the Ladoga Lake) with fictitious users in the node. The introduction of the user with low priority allows us to control water surpluses in the lakes of the Vuoksi river basin and not to let the extreme rise of their levels.Arms, connecting lakes, are conditionally presented by links 1-8 (in the comlete scheme- and 1-3 (in the simplifyed one). In computations dependencies of the surface area of lakes on the water volume have been used, and constraints on the outlet capacities of links have been introduced. These constraints have been chosen on the basis of the necessity to develop a favourable operating regime for four hydropower plants, located on the river.

Limits on changes of the active volume of lakes have been determined in accordance with the lakes' volumes for the normal headwater level and the dead volume level. Levels of desirable water stores have been chosen based on statistical processing of the observed levels for a number of years.

Depending on the problem's formulation different bounds on the system's hydrologic state and different systems of priorities have been assigned.

Initial hydrologic data have been presented as chronological series of unregulated inflows to water reservoirs. The first test computations have been carried out with the use of initial hydrologic data for 1981 year.

Next, a great number of computations, based on the rules common for the whole system, have been carried out for characteristic groups, representing 1981-1982 years and 1963-1965 years, the years of increased and lowered water bearing, accordingly. Calculations have been carried out for long-term series (1941-1982 years) also.

The carried out calculations allowed us to make the following conclusions:

- the presented mathematical model may be used for analysis of water resources system functioning of the lakes in the Vuoksi river basin;

- the selection of operating parameters of the model is relatively complex, it presupposes the researcher's good knowledge of the special features of the water resources system functioning in the Vuoksi river basin;

- to choose reliable rules of the water resources system management statistical estimation of the lake level regime should be provided (curves of the maintenance of the given level during the definite period of time, probability curves for releases into the Vuoksi river, etc.);

- the proposed mathematical model may be applied to the study of the hydrologic forecasts efficiency, allowing to develop operating rules of the water resources system of the Vuoksi river basin with account of the long-term hydrologic forecasts of flood volume.

The analysis of the forecasts efficiency for each particular water resources object is subdivided into the three main stages:

- the development of means to present forecasting data in a form, allowing one to carry out an analysis of the water resources system functioning;

- the estimation of potential expediency of the forecasts use;

- the proper study of the efficiency of the use of real forecasts with adherent errors, when the decision, made according to the forecas may not correspond with the real hydrologic state.

In the paper the results of studies, obtained for the two first stages of investigation of the efficiency of the inflow forecasts for a flood period are given.

The Use of Hydrologic Forecasts as an Operating Argument in Simulation Experiments.

Hydrologic forecasts may be incorporated in the procedure of hydrologic state determination, as the operating rules and constraints are preset for the whole period of modelling and do not depend on inflow forecasts.

The following possibilities to determine hydrologic state should be analysed to evaluate the efficiency of hydrologic forecasts:

Alternative A - for lake filling at the beginning of the calculated interval plus the inflow volume for the interval. The envisaged by this alternative advanced account of data on the future inflow volume, unknown at the time when the operating rules are being chosen, actually means the use of perfect forecasts.

Alternative B - for lake filling at the beginning of the calculated interval plus the average inflow for the interval with further use in the calculation of actual inflows.

Alternative C - for lake filling at the beginning of the calculated interval plus the forecast of the inflow volume for the interval with further use in the calculation of actual inflows.

Results of regulation based on the use of alternatives A and B characterize potential efficiency of the use of hydrologic forecasts for the given operating rules and constraints. To evaluate the efficiency of actual imperfect forecasts a new version of the model is being developed at the Institute for Water Problems of the USSR Academy of Sciences. This version will allow us to use alternative C.

Presently, long-term forecasts of the total inflow volume for a flood period to the largest lakes of the system: Saimaa, Kallavesi, Pielinen are issued in Finland. The forecasts are based on the analysis of statistical relations among the inflow and the factors, predetermining it: runoff in the preceeding period, water equivalent of snow, precipitation in the flood period. On April 1st the main forecast is issued, which is specifyed in the beginning of May and

June. Judging by the criteria, used in practice by Hydrometeoservice in the USSR, these methods may be considered to be satisfactory ones. (Table 2).

Table 2.

Accuracy Estimation of Long-Term Inflow Forecasts to the Largest Lakes of the Vuoksi River Basin for a Flood Period

Lake	Actual inflow, km^3		Forecast, km^3		Error, km^3		$\sigma_\Delta / \sigma_{actual}$
	mean	σ actual	mean	σ forecast.	mean	σ_Δ	
Saimaa	7.53	2.16	7.12	1.43	0.41	1.41	0.65
Kallavesi	2.56	0.67	2.58	0.56	-0.49	0.49	0.73
Pielinen	2.47	0.60	2.45	0.39	0.02	0.43	0.72

Possibilities to Use Operating Inflow Forecasts for a Flood Period.

The use of operating forecasts in simulation experiments necessitates to bring the length of a forecasting period in correspondence with the length of the calculated interval, used in simulation experiments. Therefore, a forecast of the flood volume should be supplemented with its seasonal distribution, in this case by monthly distribution.

Existing procedures for estimation of seasonal runoff distribution of floods are based on typifization of the observed date for one or two characteristics (e.g. the flood volume, the date of the beginning and the end of the flood period, etc.). However, consideration of any factor or a number of factors, sufficient for one case, becomes insignificant for the other, where the unaccounted hydrometeorologic characteristics will influence the runoff distribution to a greater extent. The difficulty in issuing such a forecast consists in the fact that the seasonal distribution of flood runoff to a greater extent depends on weather conditions, which could not be determined with the demanded advance, than its volume. Therefore, without long-term prediction of weather conditions deterministic means of seasonal runoff distribution prediction for a flood period will remain unreliable and practically unused in real practice.

To solve this problem it is suggested to use probabilistic means for prediction of seasonal runoff distribution of floods as conditional curves of probability distribution of a monthly inflow, developed on the basis of the long-term forecast of the flood volume and its specifications. The order of the development of curves will be illustrated on the example of the unregulated inflow to lake Pielinen.

First of all, statistical characteristics of monthly and total inflows and the pair coefficients of correlation (Table 3) are calculated to determine the most close relations. Based on the obtained results the following arguments are used in the development of conditional curves of distribution: for April - total inflow, for May - inflow from May to July, for June and July - inflow from June to July.

Table 3.

Matrix of the Inflow Correlation Coefficients, Lake Pielinen

Period	IY	Y	YI	YII	IY-YII	Y-YII	YI-YII
IY	1						
Y	0.11	1					
YI	-0.04	0.49	1				
YII	0.05	0.43	0.65	1			
IY-YII	0.37	0.82	0.76	0.75	1		
Y-YII	0.06	0.85	0.82	0.78	0.95	1	
YI-YII	0.00	0.50	0.91	0.90	0.83	0.89	1

On the scheme of the relation of the monthly inflow values and the chosen argument the points are distributed along the regression lines, being the locus of conditional mean values under fixed value of the argument. The lack of a significant relation between values of deviation from the regression line and the argument allows one to consider parameters of unconditional distribution of the pointed deviations to be parameters of conditional reliability curves of a monthly inflow, being near to normal (C_S in all cases is near to zero). From Table 4 we can see that the probabilistic forecasts are expedient for May, June and July. Unsufficiently close relation for the development of conditional curves of the April runoff to the total runoff of the flood period is caused by uncoincidence of the beginning of actual flood period with the one fixed in calculations.

Table 4.

Parameters of Unconditional and Conditional Distribution of Inflows for the Flood Period, Lake Pielinen

Month	Mean, km^3	σ uncondition. km^3	σ conditional	σ cond./σ unc.
April	0.410	0.195	0.181	0.93
May	1.050	0.315	0.166	0.53
June	0.560	0.205	0.085	0.41
July	0.396	0.193	0.084	0.44
April-July	2.417	0.630		
May-June	2.006	0.587		
June-July	0.956	0.362		

Therefore, conditional curves of reliability, developed for a particular month, represent a family of parallel straight lines, differing only by conditional mean values,which depend on the argument values (Fig. 2). It should be noted that the described method of the development of conditional curves can be applyed only to conditional distribution with moderate assymethry and absolute values, significantly greater than zero. And in the opposite case normal curves will incorporate negative inflow values, contradicting with its physical essence. In the pointed cases it is expedient to carry out some preliminary normalization of initial data, allowing to normalize initial distributions.

On the second stage of investigations "basic" operating rules should be developed, using perfect forecasts as a basis to evaluate the efficiency of operating forecasts (the third stage). As for the possibilities of the discussed above model the second-stage studies have been conducted for alternative A. Under such conditions with the flood forecast for the three main lakes of the system (Saimaa, Kallavesi, Pielinen) the operating rules have been developed for a simplifyed linear-nodal scheme (fig. 1b). Under simplification only 8-9% of the total inflow to the system is not taken into account, which is quite permissible. The limits on outlet capacities of water ways interconnecting lakes are assigned with account of simplifications of the initial scheme (Table 5).

The simulation experiments,aimed at the development of operating rules, have studied various variants of lines' combination of the desirable lake filling at the end of the month and a system of priorities.

It has been founded that, if for the wet hydrologic state high lines of filling are desirable and for the dry state - low lines are desirable, and the priorities of upper lakes, compared with the priority of the lake Saimaa are lower, the upper lakes, being drawn-down to the lower levels in low flow periods with the future low inflow values, turn inappropriate to provide even minimal acceptable limits of flow in the channel to the Saimaa lake due to their extremely low filling. In a high-flow period the situation changes to the opposite - the lakes Kallavesi and Pielinen, filled to the normal headwater level are not able to accumulate the unregulated inflow, as they have not been drawn down in advance with the high levels of the desirable filling in a high-flow period. Therfore, it has been decided that the lakes Kallavesi and Pielinen should be filled to the highest possible level in the low flow period and compulsory draw-down before the period of high runoff should be done. The pointed condition is implemented by alteration of the order of the lines' assignement of the

desirable filling of the upper lakes (the mean hydrologic state corresponds to 10% filling, the wet and dry states - to 50% filling) and a system of priorities for water resources system users (Table 6), guaranteeing the surplus water discharge into the Ladoga lake, when the Saimaa lake is filled higher than the average level. A comparison of natural and calculated levels of lake filling shows that the use of perfect forecasts of monthly inflows furthers the decrease of the amplitude of lake level fluctuations (Fig. 3).

Table 5.

Paramaters of the Simplifyed Linear-Nodal Scheme (Fig. 1b), the Vuoksi River Basin

Node No., link No.	Lake filling, thousand m^3		Constraints for links, m^3/s	
	initial	for NHL	upper	lower
1	5498	11400	490	50
2	435	939	280	95
3	558	2218	850	300

Table 6

The Resulting System of Priorities of the Water Resources System Components

WRS component	Priority in hydrologic state		
	dry	mean	wet
Saimaa	6	40	75
Kallavesi	5	40	70
Pielinen	5	40	70
Ladoga	72	72	72

The obtained results form the basis for the study of the forecasts efficiency for the unregulated inflow to lakes of the Vuoksi river basin. It should be held in mind that the discussed above results characterize potential efficiency of the forecast use, as they are obtained without account of forecasting errors of the inflow volume for a flood period and for the chosen probabilistic model of seasonal inflow distribution. The task of the next stage of studies is the development of such a technology of the forecasts use, which might allow us to use real possibilities of runoff prediction most fully, meeting the demands of the chosen constraints on the outlet capacity of water ways and the level regime of lakes.

Conclusions:

1. The carried out verification of the model's functioning for a system of lakes of the Vuoksi river basin has shown that it can be used to solve a number of problems, connected with the development of operating rules of the pointed system.

2. The use of the model presupposes the good knowledge of the specific features of the given water resources system functioning; without it the work with the model's operating parameters in simulation experiments is not possible. At the same time the work

with the model results in the increased knowledge of the researcher on the studied object, reflecting some specific features of the system's functioning under various alternatives of operation, which can be furter used for economic analysis, in particular, based on the use of the given data on damages for branches of national economy under violation of desirable levels of lake fluctuations in evaluation of the operating rules efficiency.

4. The analysis of the forecasted and actual runoff series to the lakes of the Vuoksi river basin for the period of spring flooding has shown that the forecasting methods used at the present time have an acceptable accuracy, allowing to increase the efficiency of the Vuoksi system's management. Future studies should be aimed at the development of probabilistic models of seasonal distribution of the forecasted inflow.

5. The obtained results of modelling of the system's functioning based on the use of perfect forecasts show their high potential efficiency. At the same time the rules of the real forecasts use should take into account both possible forecasting errors and possibilities to correct operating decisions according to forecast specification. The new version of a mathematical model, being developed for realization of these statements, should incorporate in determination of hydrologic state forecasts and their specification and should have a possibility to adjust to different management algorithms during forecast specification.

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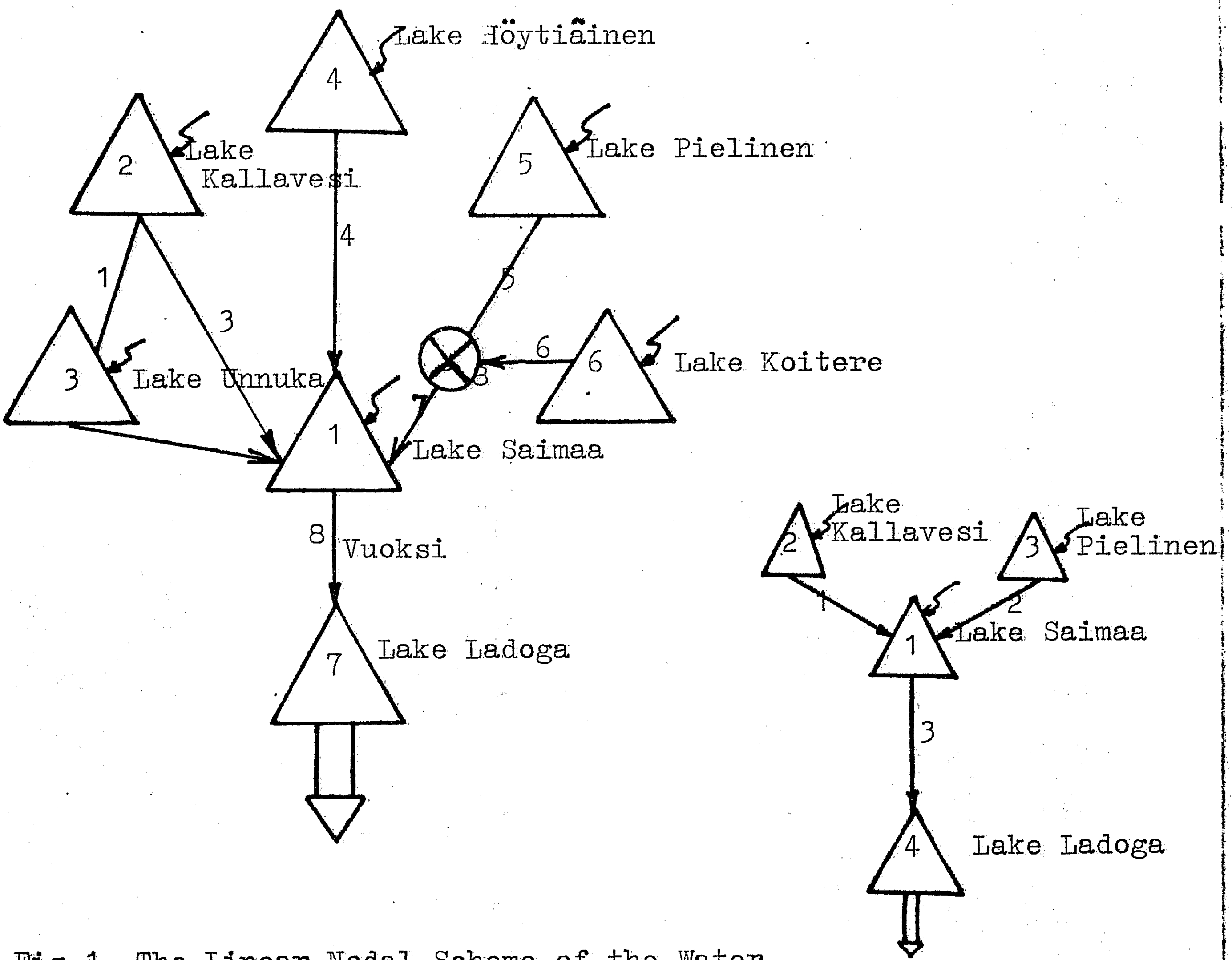

Fig.1. The Linear-Nodal Scheme of the Water Resources Systems of the Vuoksi River Basin

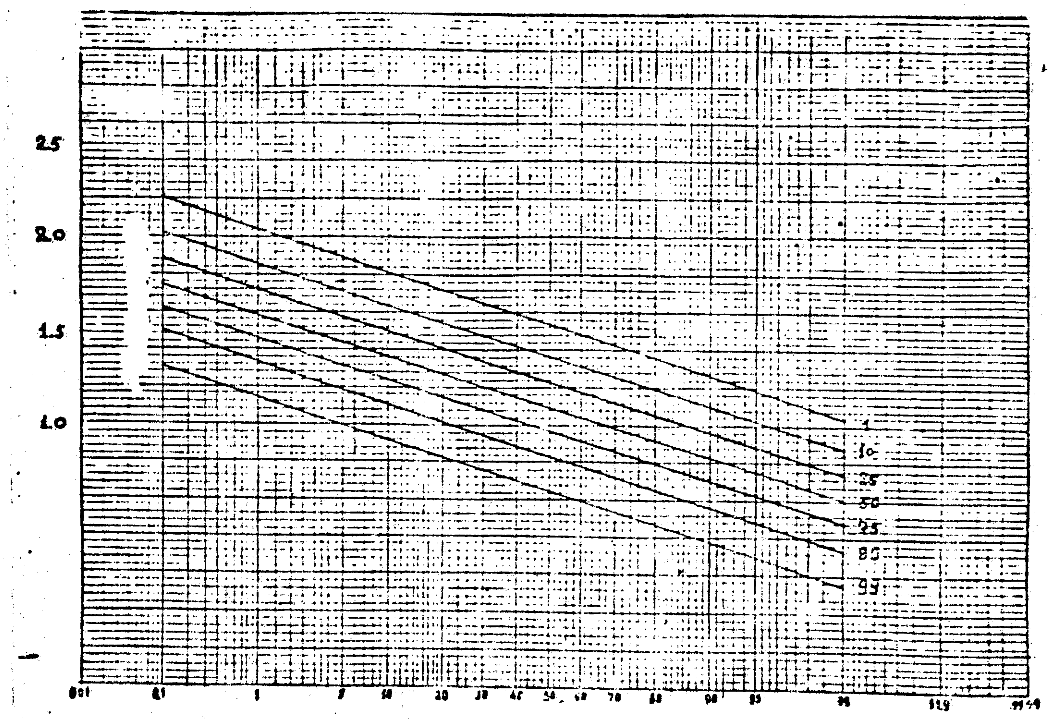

Fig.2. Conditional Probability Curves of Inflow, Lake Pielinen (May)

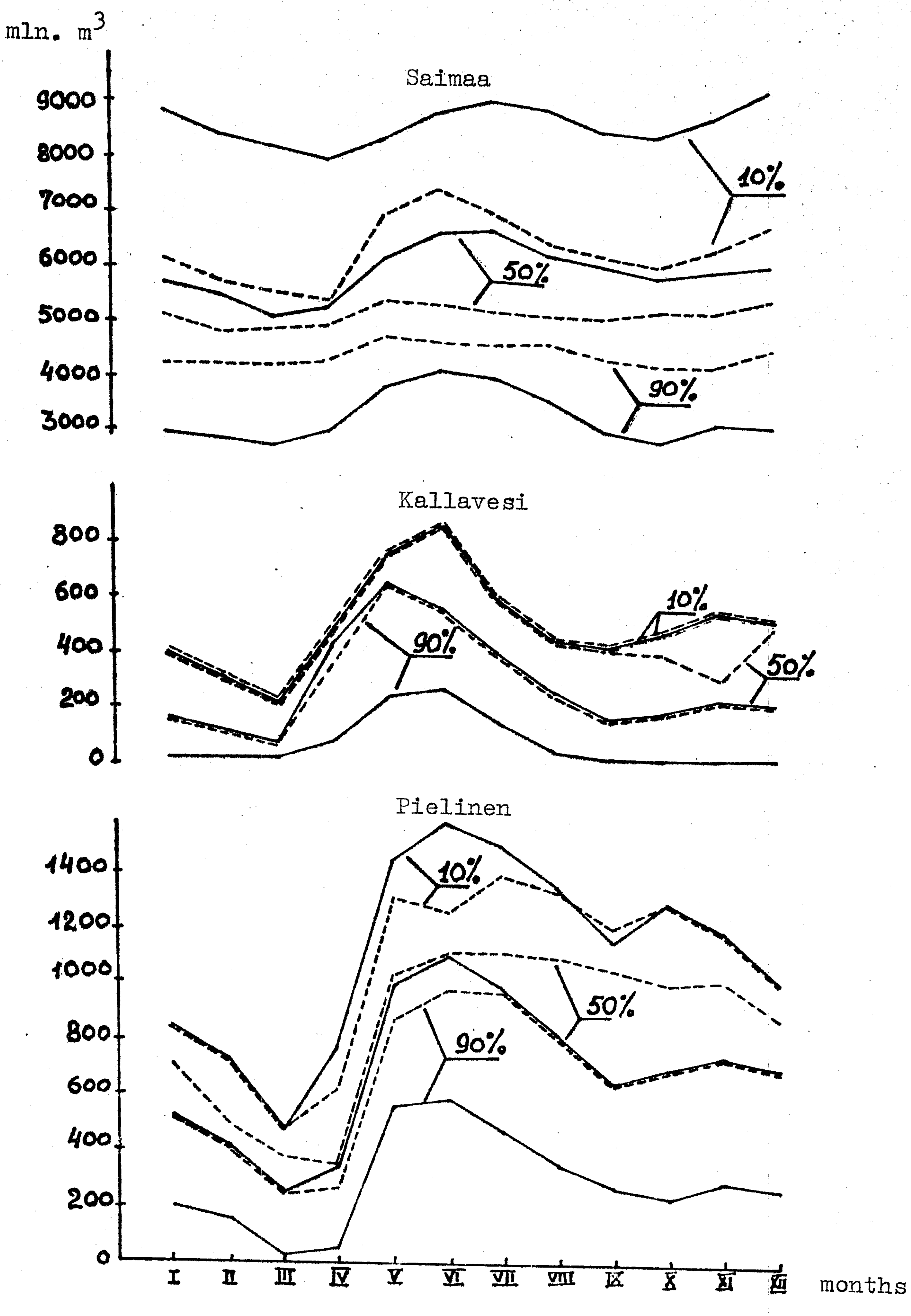

Fig.3. Actual (Continuous Line) and Calculated (Dotted Line) Equally Probable Fillings of the Lakes. Line Numbers Stand for Reliability of Filling

THE USE OF RAINFALL-RUNOFF MODELS IN REAL TIME FORECASTING AND CONTROL

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INTRODUCTION

Studies on rainfall-runoff modelling have been carried out in the Laboratory of Hydrology and Water Resources Engineering, Helsinki University of Technology since 1978. Vakkilainen and Karvonen (1980) and Karvonen (1980) have studied the use of the SSARR model in a research programme which also included the application of linear rainfall- runoff models (Järvinen, 1982) and snowmelt-runoff models (Hiitiö, 1982). Nyrhinen (1982) and Rajantie (1986) have studied the applicability of the Swedish HBV-model. Vakkilainen and Karvonen (1982) have developed the SATT model. In this model model the soil moisture can be treated in a physically based way or using a conceptual approach. The flood routing, as well, can be calculated either by using the Saint Venant equations or the Muskingum method. The model includes a simple adaptive system which improves the accuracy of the forecasts when new measured data is available. Karvonen (1985) formulated both the HBV model and the SATT model within the framework of state-space analysis and used the extended Kalman filter for improving the forecasts.

For the optimal control of a watercourse an optimization method is needed, too. Both linear and dynamic programming models have been developed for operating a single lake or reservoir (Vakkilainen, 1978; Heikkinen, 1982).

The rainfall-runoff models mentioned previously have proved to be either too complicated to use together with control methods or the correction capability of these models is insufficient. Hence, the applicability of transfer function models has been evaluated (Malve, 1986). It is also difficult to use linear and dynamic programming techniques for cotrolling a multi-reservoir systems. Therefore, a method based on the Pontryagin maximum principle has been recently developed (Karvonen, 1986). In this paper these two studies have been shortly reviewed.

REAL TIME FORECASTING

The purpose of the work of Malve (1986) was to combine and test conceptual rainfall-runoff model and time series models (AR-models and transfer function models). Model combinations were simple in order to obtain suitable models for real time foreacasting. The main emphasis was to compare the predictive ability of three different models:

1) HBV-model
2) HBV-model + AR-model
3) Transfer function models

The AR-model used together with the HBV-model was aimed at modeling the residual between the measured and forecasted discharge, i.e. the residuals of one or two previous days were used to correct the forecasted values. In model version 3) the rainfall losses were estimated using the soil moisture storage of the HBV-model.

The model combinations were tested using observations from four catchments in western and northern Finland: Tujuoja ($A=21$ km²), Yläneenjoki (195 km²), Loimijoki (1980 km²) and Ounasjoki (12300 km²). Transfer function models proved to be the best ones (Malve 1986). The AR-model improved clearly the short-term forecasting capability of the HBV-model. A brief review of the results is shown in Table 1.

Examples of the final equations of model version 3) for the Loimijoki cathcment are:

$$Q(k+1)=0.6477*Q(k)+0.0935*P(k)+0.0923*P(k-1)+1.3677*Y(k)$$

$$Q(k+1)=0.6374*Q(k)+3.1691*Y(k)$$

where $Q(k+1)$ and $Q(k)$ are discharges at time $k+1$ and k, respectively. $P(k)$ and $P(k-1)$ are the effective rainfall rates at two previous days and $Y(k)$ is the discharge of a nearby smaller reference catchment.

In Fig. 1 a comparison of the calculated and measured discharges using model versions 1) and 3) in the Ounasjoki watershed is presented.

Table 1. The comparison of the results of different models

Watershed	Model 1)	Model 2)	Model 3)
Tujuoja	0.85	0.93	0.93
Yläneenjoki	0.79	0.87	0.91
Loimijoki	0.84	0.92	0.93
Ounasjoki	0.81	0.98	0.99

The goodness of fit is calculated using the equations:

$$RR = (F0 - F)/F$$

$$F0 = (Q(i) - QM)^2$$

$$F = (Q(i) - QC(i))^2$$

where QC(i) and Q(i) are calculated and measured discharge, respectively, and QM is the mean of measured discharges.

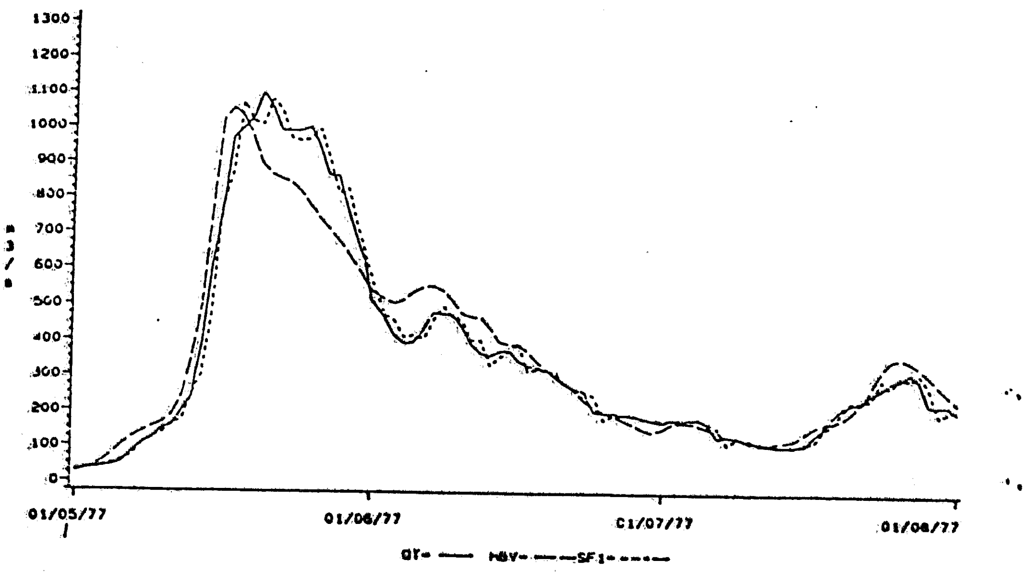


Fig. 1. Measured and calculated discharge in the Ounasjoki catchment area. QT is measured discharge, HBV and SF1 are calculated discharges using the HBV-model and transfer function model, respectively.

REAL TIME CONTROL OF MULTI-RESERVOIR SYSTEM

The optimal control system is based on the Pontryagin maximum principle (e. g. Kalaba, 1982). The general idea of the maximum principle is to maximize (or minimize) the value of the selected cost function without violating the constraints (maximum and minimum discharge and water level). The cost function can be e.g. to maximize the energy product of the reservoir system or to minimize the outflow of the reservoir system in order to prevent damages caused by floods.

The Kalajoki reservoir system (see Fig. 2 and 3) has been used as a testing example. The total area of the watershed is 4 200 km^2. In the area there are five regulated reservoirs which can be used for real time control of river flow. Inflows into each reservoir are predicted using transfer function models. Two types of forecasts are used. First, short-term forecast for 5 to 10 days are calculated using predicted air temperature and precipitation forecasts. Second, the total volume of the spring flood is estimated using meteorological data of previous years.

A model of the reservoir system is needed in the optimization. The mathematical model of the reservoir system is composed of waterbalance equations and transfer function models.The last mentioned are used to describe the flow in rivers between the reservoirs. In the optimization the goal is to minimize outflows from the reservoirs.

The general idea in real time operation of the reservoirs can be summarized as follows:

1) Short-term and long-term inflows are forecasted. The long-term forecasts (total inflow volume) are needed to prevent the filling of the reservoirs too early.

2) The Pontryagin maximum principle is used to obtain optimal outflow trajectories for each reservoir for the whole operating period (e.g. 2 weeks - 2 months).

3) The operation policy is fulfilled until new measurements are available.

4) These new measurements are used to update the inflow forecasting

models. The procedure is repeated from the step 1).

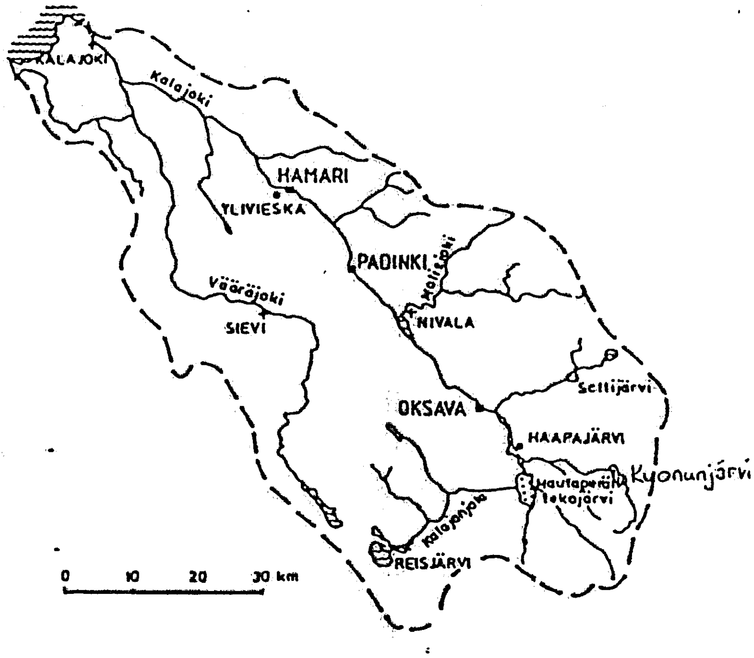
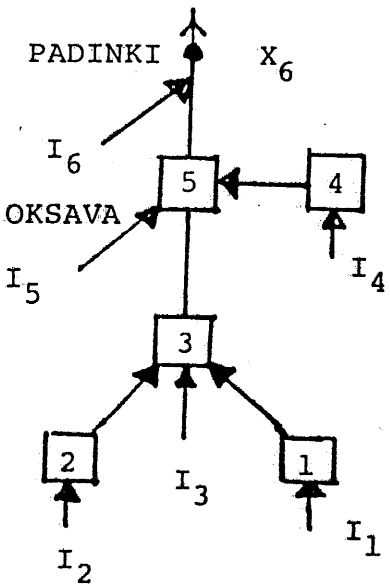


Fig. 2. The Kalajoki watershed area.



Nro	Allas
1	Kuonanjärvi
2	Reis-Vuohtojärvi
3	Hautaperä
4	Settijärvi
5	Haapajärvi

Fig.3. The reservoirs of the Kalajoki watershed area.

An example of the optimized outflow trajectories are presented in Fig. 4. In this figure the inflows into each reservoir are shown, too.

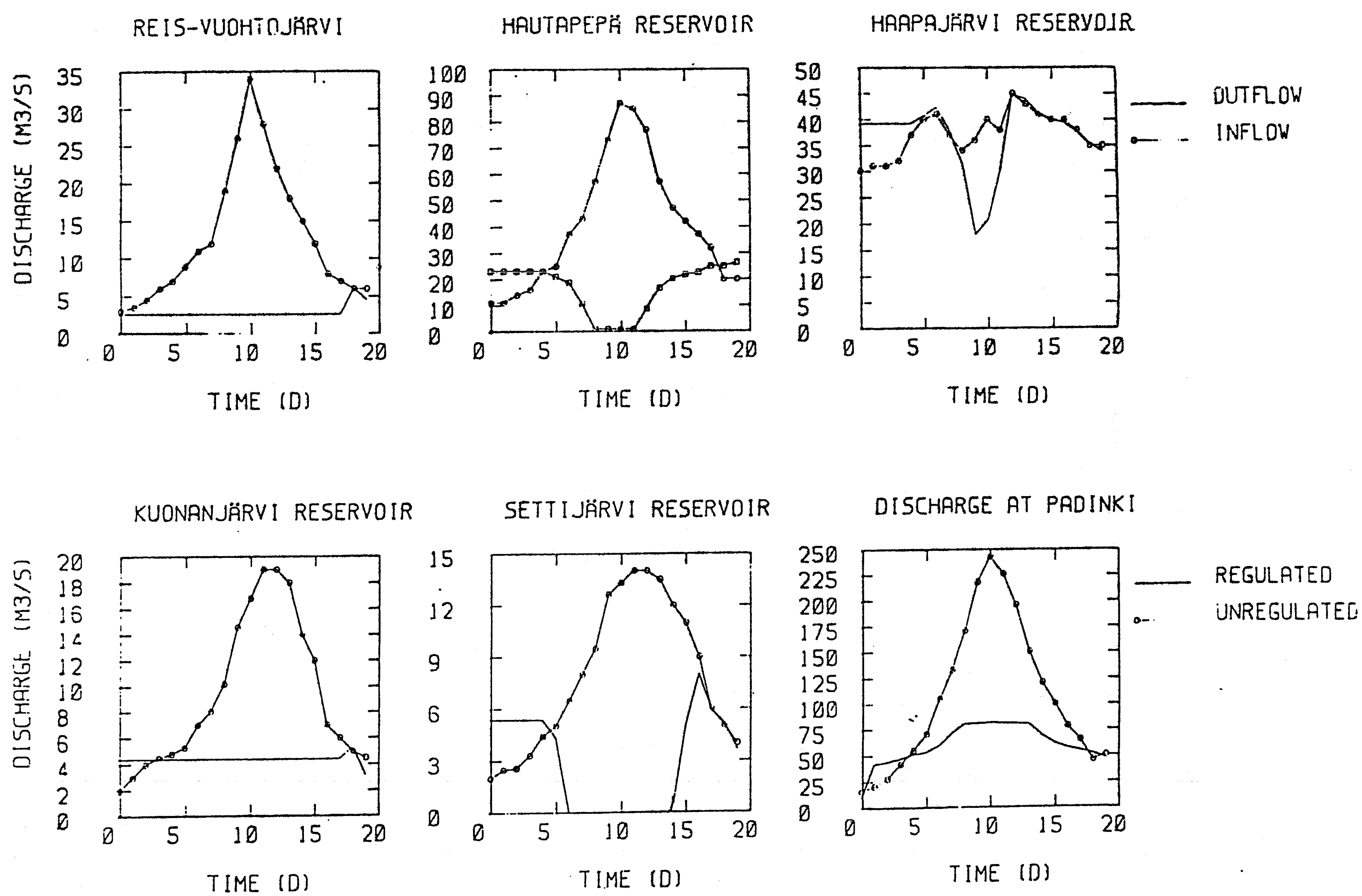


Fig. 4. Optimal outflow trajectories for each reservoir and the regulated discharge at Padinki.

CONCLUSIONS

Transfer function models have proved to be a powerfull tool in real time flood forecasting. In the context of these type of models it is easy to accomplish the updating of the flow forecasts using the latest hydrological data. Moreover, the information of the experimental and representative basins can be included in the forecasting model.

The Pontryagin maximum principle seems to be applicable to optimal control of regulated reservoirs. The formulation of the objective function is perhaps the most difficult part of the optimization procedure.

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MODELS OF SNOWMELT RUNOFF FORMATION IN THE FOREST ZONE AND THEIR APPLICATION TO HYDROLOGICAL FORECASTING

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According to the usual terminology, hydrological forecasting is regarded only as a prediction of hydrological characteristics, based on the use of physical laws, governing the development of hydrological processes, and on hydrometeorological data - observed at the moment of the forecast preparation or reliably forecasted. It is important to emphasize the difference between forecasting and statistical prediction: the latter takes into account only the natural variability of hydrological values for a long period of time. The forecasts of runoff is the most valuable hydrological information, that can be used to control water resources systems (WRS) in real time scale. Theoretically, lead-time of these forecasts depends on the inertion of hydrological systems and possibilities of forecasting meteorological data. However, the observation data and the quality of mathematical models are no less important. Automatization of observations of hydrological and hydrometeorological regime in the river basin, the development of automatic systems of data collecting and processing have considerably increased the possibilities of hydrological forecasting and its application to WRS control.

Considerable success has also been achieved in applying mathematical models for the description of the main processes determining forecasted characteristics. But the level of these models themselves and the range of their application to the forecasting of snowmelt runoff cannot yet ensure the possibilities provided by the limits of natural predictabilty of runoff parameters (even if we do not take into account our possibilities to forecast input hydrometeorological data), and the state-of-the-art of present methods of hydrometeorological measurements and information transmission.

The formation of plain river snowmelt runoff is a complex interaction of many hydrological processes, depending mainly on geographical, climatic and soil peculiarities of the river basin. The most complete possible account of these peculiarities allowed us to find a simplified structure of the model, including characteristics of highest prognostic significance. Composition and role of such characteristics can vary depending on the lead-time and types of forecasts.

In many cases the choice of such characteristics can be based on the data of experimental observations. However, owing to the complex character of processes and great variability of hydrometeorological processes, conclusions based on the experimental data, can prove unreliable. Numerical experiments using physically based models of the main processes seem to be helpful in determining the most informative predictors.

In a general way, the peculiarities of snowmelt runoff formation in northern areas of the forest zone, envelopping Finland and the northern part of the European part of the USSR, may be formulated as follows. Soils in this region have high permeability. That is why overland flow is seldom observed here, and runoff losses weakly depend on hydrothermal processes in the soil. Water content of the soil is most important in forming runoff losses; surface water detention in swamps and litter is considerable. The greater part of water flows as a subsurface runoff after snow melting is over. But it should be emphasized that these peculiarities manifest themselves as usual, whereas some years and on some watersheds the main processes can occur in quite the opposite way.

As a result of the investigations carried out at the Water Problems Institute we developed a detailed physically based model of snowmelt and rainfall runoff formation. This model allows us to describe the processes occurring in various geographical zones. It takes into account spatial development of the processes, taking place during snow cover formation, snow melting water infiltration into frozen and thawed soils, surface and subsurface flow, evaporation and interaction of surface and ground waters. The results of the model testing were described in the Proceedings of the International Association of Hydrological Sciences (IAHS Publ. No. 155, 1986).

The use of such complex model for short-term operative forecasts is yet premature, even if we have all the necessary input data. The main peculiarities for short-term forecasting for management problems in real time scale is the possibility to use in simulation processes comparatively short observation series of forecasted and input values. Another characteriristic feature is the possibility to correct the forecasts themselves as new data appears. (Such process is called updating). In many cases it allows us to use rather simple conceptual models for operative forecasting. One of the models, having a number of common presumptions with the one mentioned above, has been developed for the forest zone by N.A. Nazarov at our laboratory.

In this model the intensity of snow melting is estimated by the snow water equivalent and degree-day factor separately for forest and field areas of the watershed. The intensity of water freezing in snow during cold snaps, the spatial distribution of snow during its melting as well as the dynamics of snow density and its water holding capacity are also taken into account.

Hydrological Regime of frozen and thawed soils is described by a balance model of vertical moisture movement where soil absorption capacity is

$$I(t) = \begin{cases} \frac{K_p}{(1+8\cdot\Lambda)^2} \left(\frac{P-\Lambda-\phi_o}{P-\theta_o}\right)^4, & QB > I \\ QB, & QB \leq I \end{cases}$$

The ice content of soil is

$$\Lambda(t) = \begin{cases} \frac{W_1}{\Delta Z} - \phi_\kappa, & W_1 > \theta_\kappa \cdot \Delta Z \\ 0, & W_1 \leq \theta_\kappa \cdot \Delta Z, \end{cases}$$

where $QB(t)$ is intensity of water input;

W_1 is the total volume of meltwater and ice in the soil layer ΔZ; K_p is filtration coefficient; P is the porosity of soil; θ_o and θ_κ are the volumetric content of fixed and non-frozen soil moisture.

The intensity of effective water inflow to the watershed surface is determined separately for field (f_s) and forest (f_f) areas:

$$Y(t) = (1-f) \sum_{k=1}^{2} \phi_k \; (QB_s + R - I_{s,k}) \; (1 - e^{-SP\int_0^t (QB_s + R - I_{s,k})d\tau}) + f \cdot$$

$$(QB_f + R - I_f) \cdot (1 - e^{-SP\int_0^t (QB_f + R - I_f)d\tau})$$

If $QB + R \leq I$, than $R + QB - I = 0$; R is the average watershed intensity of return flow, caused by the difference between water discharge in the subsurface storage and its discharge capacity; f is the coefficient, reflecting what part of the watershed area is covered with forests; SP is the parameter of surface water interception; is a part of field watershed area, occupied by frozen (K=1) thawed or weakly thawed (K=2) soils, the depth of freezing H=20 cm. The ϕ_1 ($\phi_2 = 1-\phi_1$) value is determined by a two-parametrical γ-distribution depending on the relative depth of freezing and variation coefficient $C_v = C_v\ (H_H)$ over the area. The depth of freezing H is calculated using the approximate analytical solution of the problem of heat transfer, taking into account the warming effect of near-surface ground aquifers on the frozen soil. The character of snow cover temperature variation is assumed to be almost linear in frozen and thawed soils. The coefficient C_v is assigned on the basis of generalized relationships, received at the State Hydrological Institute on the basis of observation data for the northern area of the European USSR.

The average watershed intensity of runoff-forming inflow to the subsurface storage is:

$$W(t) = ((1-f) \sum_{k=1}^{2} \phi_k \cdot P_\kappa + f \cdot P_3) \cdot S_1,$$

where

$$P(t) = \begin{cases} I(t), & \theta_\kappa > \theta_{\kappa B} \\ 0, & \theta_\kappa \leq \theta_{\kappa B} \end{cases}$$

is the volumetric moisture, corresponding to the minimum water holding capacity of the soil; S_1 is a dimensionless parameter. The transformation of the average watershed effective inflows Y(t) and W(t) into the hydrograph is carried out using the equations of kinematic wave or the Duamel integral separately for surface and subsurface flow.

The side inflow model was verified for five separate watersheds in the Sukhona river basin.

A model, structurally close to the above mentioned ones, was proposed by V.I. Koren and V.A. Belchikova /2/ and applied to operative short-term forecasting of snowmelt runoff in the forest zone, (in particular, for the basin of the Yug river). A simplier description of losses formation process but with a greater degree of detalization of snow melting is presented in the work of B.Veviläinen and Yu. Motovilov, submitted to the Symposium.

Analysis of the results of these works and the investigation of short-term forecasts of snowmelt runoff for some other geographical zones was carried out. (Here we should mention the Project of the World Meteorological Organization on the Intercomparison of Conceptual Models of Snowmelt Runoff, where 11 different models were presented /3/). It allows us to conclude that further increase in accuracy and reliability of short-term forecasts should be carried out not so much by way of sophistication of the models, but by way of improving updating. In forecasting we can correct only some of model values, such as precipitation, water discharge or snow cover distribution. It is important that in the process of updating the information, received on the basis of long-term observations, would not be changed (as it is done in some works) - only initial values of variables, calculated in the process of forecasting, could be changed. Therefore, the model which is capable of proper updating will have advantages. Another important way of improving short-term forecasts is the search for new types of information for updating. Such a possibility can be provided by using aerocosmic and other remote sensing methods of measurements.

As far as the development of long-term (2-3 months) forecasts technique is concerned, the situation here is quite different.

At present long-term forecasts are based on statistical dependences where snow water equivalent and various indices of soil absorption capacity are used as predictors. Spring precipitation is assumed to be of minor importance or to remain practically unchanged from year to year.

This approach has two serious drawbacks:

1. The received dependences reflect the average peculiarities of runoff formation, whereas unusual situations are most important in forecasting. In particular, it turns out that for the forest zone it is usually impossible to establis significant statistical correlations between the volume of spring flood and the depth of soil freezing (or any other indices, denoting variation of soil absorption capacity due to ice content). At the same time we can consider that in certain years this effect can play an important role in runoff formation.

2. Indices, used for establishing statistic dependences, can only characterize soil processes in the indirect manner. Thus, the most frequently used indices of soil absorption capacity are the sums of differences between precipitation and evaporation for the three months, before the establishment of the snow cover, as well as the depth of soil freezing. Such indices, however, do not take into account vertical moisture distribution and its movement in the soil, particularly during freezing processes, that can entail considerable changes in the moisture content of the upper soil layers.

The detailed physically based model is assumed to be helpful in eliminating the above mentioned drawbacks without the considerable increase in the amount of input data. We learned (on the example of the Kostroma river basin), that even a simplified physically based model of moisture movement and soil freezing can provide more informative indices for soil absorption capacity than the commonly used statistical relationships /1/. It can also be expected that even the preservation of the hypotheses of weak influence or stationarity of meteorological factors in spring could ensure a more reliable determination of runoff values, if physically based models of snowmelt runoff are used. But these possible precisions do not mean any radical changes in the methodological basis of long-term forecasts of spring runoff. And we think that the necessity for such radical changes has already become urgent. They can considerably increase the efficiency of long-term hydrological forecasting in WRS management. Such changes can be realized through developing dynamic-stochastic models of snowmelt runoff formation, as well as through probabilistic forms of forecasts. (Attempts are being made in this sphere, based on the stochastic description of forecast errors, not on the account of stochastic character of the processes themselves). The Programme can be as follows.

1. The use of physically based deterministic models for the processes of snowmelt runoff formation.

2. The use of the Programme seems to be most difficult. Spring precipitation can vary within a wide range and its correlation for certain calendar periods are usually weak. (Five-or ten-day time interval can be used for this purpose). It is still more difficult to develop air temperature models, containing sufficient information; (a time interval of 24 hours should be used here). However, high accuracy is not to be achieved here, since the influence of snow melting intensity on the runoff in the forest zone of the watershed is considerably less, than in the field ones.

3. The determination of possible runoff volumes or maximum discharges, as well as quantiles of distribution of these values by the Monte Carlo method. Taking into consideration that in hydrological forecasting we are mostly interested in the average expected values, the amount of numerical experiments can be small.

4. The degree of complexity of physically based and stochastic models can be determined by estimating variations of entropy or quantity of information, causing different levels of the model sophistication.

Such an approach is assumed to ensure the possibility of optimum utilization of the available information for improving the quality of forecasts.

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SNOW COVER AND SNOWMELT RUNOFF MODEL IN THE FOREST ZONE

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Conditions of snowmelt runoff formation are similar in the watersheds of Finland and in the north-western part of the USSR forest zone. In operative hydrological practice of the Finnish National Board of Waters a modification of the conceptual model HBV-3 (1,4) is used for calculations and forecasts of spring-flood hydrograph. Within the framework of Soviet-Finnish scientific co-operation investigations are carried out to improve this version, so that it could later be used for the evaluation of the influence of man-induced factors on the runoff in forest watersheds. At the first stage model blocks, deseribing snow cover formation and snow melting are improved. The paper presents the main model algorithms and the results of its testing in one of Finnish watersheds.

1. General structure of the model

The model contains a simplefied description of the following processes: snow cover formation and snow melting, infiltration and accumulation of meltwater in the soil in the aeration zone, formation of surface, subsurface and groundwater runoff (see Fig. 1).

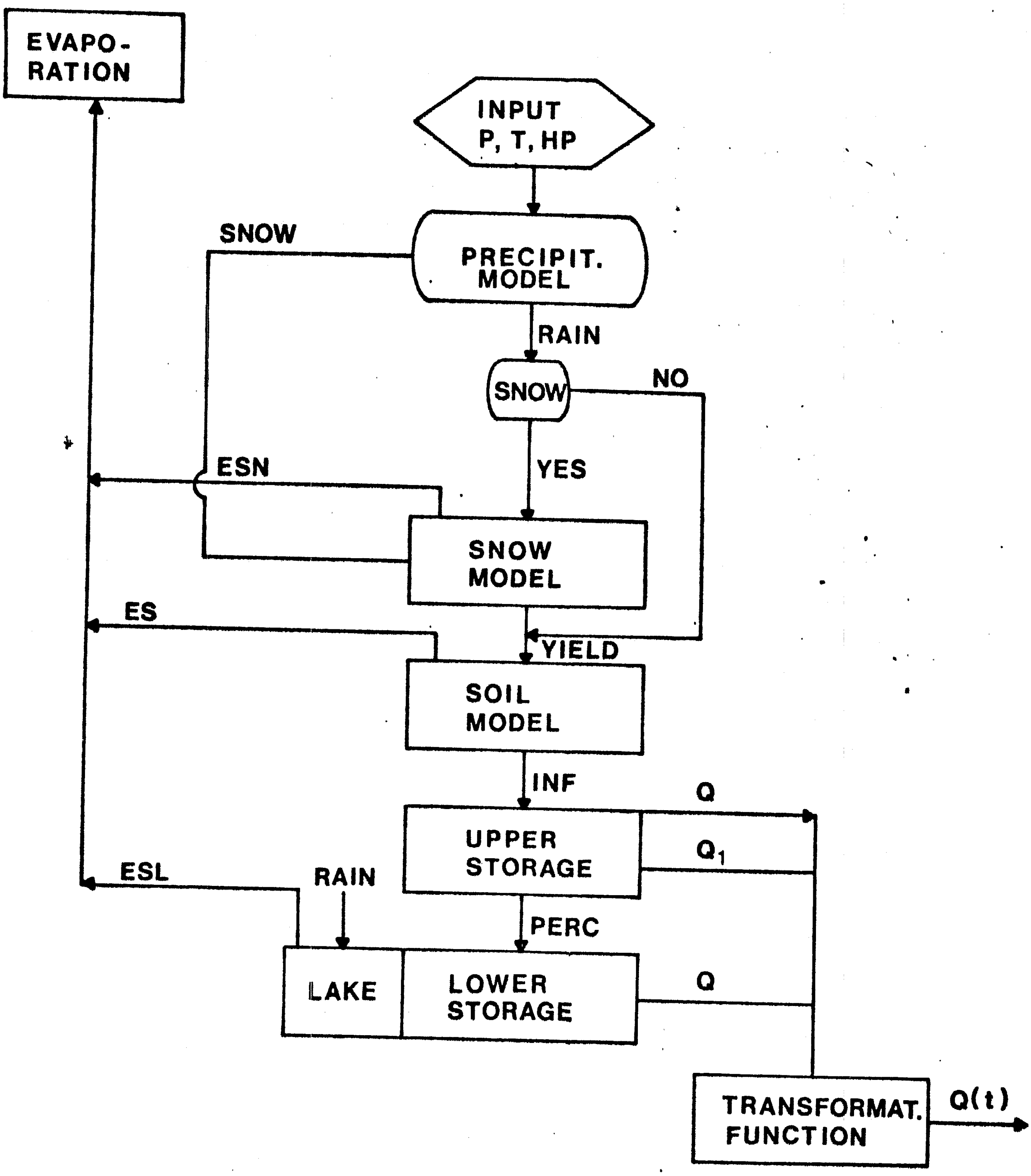

Figure 1. General structure of the model.

1.1 Precipitation

Phase composition of precipitation was determined by the daily average air temperature TATM. It was assumed, that at TATM < TMIN only snow falls, at TATM > TMAX - only rain and within the interval TMAX > TATM > TMIN both snow and rain fall, and their correlation being determined by linear function on air temperature. TMIN and TMAX values can be taken from meteorological observation data or selected during the process of calibrating model parameters. In order to account the errors in measuring precipitations, arising from evaporation, blowing off from rain gauge and retention on crowns of forest, the amount of snow precipitation was multiplied by a specific correction coefficient 1.3. for rainfall this coefficient was equal to 1.1.

1.2 Snow cover

In the original HBV-3 version snow water equivalent was calculated by summing the amounts of snow precipitation. A method of temperature index is used to calculate snow melting. The model block, describing snow cover formation and snow melting was changed in order to improve it and will be presented in detailed form below.

1.3 Soil moisture content (MVS)

Soil moisture content in the aeration zone is determined from balance equation

$$\frac{d\ MVS(t)}{dt} = YIELD\ (t) - ES(t) - INF(t) \qquad (1)$$

where

$$ES(t) = EP(t)\,\frac{MVS(t)}{LP}\ ;\ INF(t) = YIELD(t)\left(\frac{MVS(t)}{FC}\right)^{x};$$

YIELD is the intensity of inflow of meltwater on the watershed surface; ES is evaporation intensity; INF is the intensity of water inflow into the upper soil zone (this zone may be regarded as the volume of non-capillary macro-pores in aeration zone, where subsurface flow can be generated);

EP is potential evaporation; LP is soil moisture content after which evaporation achieves its maximum; FC is possible maximum soil moisture. (The physical sense of this value is close to that of soil moisture content under field capacity); x is a parameter; t is time.

1.4 Upper zone (Fig. 2)

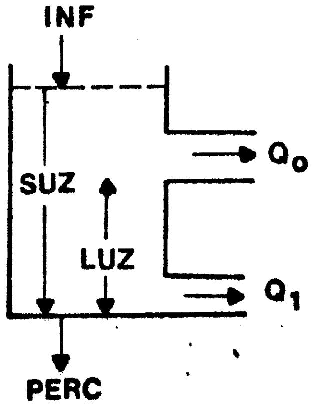

Figure 2. Upper zone.

Water storage in the upper zone (SUZ) is calculated according to

$$\frac{d\ suz(t)}{dt} = INF(t) - Q_o(t) - Q_1(t) - PERC \qquad (2)$$

where $Q_1(t) = K_1\,SUZ(t)$;

$$Q_o(t) = K_o UZ(t);$$

$$UZ(t) = \begin{cases} SUZ(t) - LUZ & \text{for } SUZ(t) > LUZ, \\ 0 & \text{for } SUZ(t) < LUZ; \end{cases}$$

PERC is the intensity of water inflow to the lower zone; K_1, K_o are parameters.

Physical sense of Q_1 value is close to that of subsurface runoff. LUZ can be regarded as the maximum volume of non-capillary macropores in aeration zone; after filling these macropores begin fast flow Q_o.

1.5 Lower zone (Fig. 3)

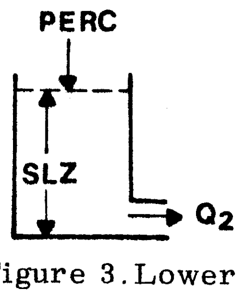

Figure 3. Lower zone.

The balance equation for the lower storage is as follows

$$\frac{d\ SLZ(t)}{dt} = PERC - Q_2(t) \qquad (3)$$

where $Q_2 = K_2\,SLZ(t)$ is groundwater runoff.

1.6 River runoff

Water discharge in the outlet point in the basin Q is calculated as

$$Q(t) = KR\ Q_3(t) + (1 - KR)\left[Q\ (t-\Delta t) - Q_{min}\right] + Q_{min} \quad (4)$$

where $Q_3 = Q_o + Q_1 + Q_2$; Q_{min} is the minimum water discharge; KR is a parameter.

2. Snow cover during its formation and snow melting

Snow cover characteristics undergo temporal variations, caused by precipitation, snow melting, freezing of melt-water in snow and snow compaction.

2.1 Account of precipitation

If snowfall occurs, the depth of snow cover H increases by ΔH_p for time interval Δt, and

$$\Delta H_p = \frac{PSN \cdot RW}{RN} \quad (5)$$

where PSN is layer of snow, fallen during Δt period (in water equivalent); RN is density of new snow; RW is density of water.

The average snow density (DSN) in this case is calculated using the condition of mass conservation

$$DSN = \frac{DSN'\ H' + \Delta H\ RN}{H' + \Delta H_p} \quad (6)$$

Here and below a touch indicates snow characteristics at the moment of time $(t-\Delta t)$, disregard of the process under consideration (in this case H' is depth of snow before snowfall at the moment $(t-\Delta t)$.

If rainfall occurs, the part of rain (PLIQ) can be retained in snow under the influence of capillary-sorption forces of snow. We assumed, that if the precipitated rain cannot fill the whole snow layer, so that its maximum water-holding capacity is achieved, all the rain is retained by snow. Else surplus rain water flows into the soil surface (water yield of snow is observed). This assumption can be expressed in a mathematical form:

$$W = \begin{cases} (W'H + PLIQ)/H & \text{at} \quad PLIQ < (WC - W')H; \\ wc, & \text{at} \quad PLIQ \geq (WC - W')H; \end{cases}$$

$$YIELD = \begin{cases} 0, & \text{at} \quad PLIQ \leq (WC - W')H; \\ PLIQ-(WC-W')H, & \text{at} \quad PLIQ > (WC - W')H; \end{cases} \tag{7}$$

$$DSN = DSN' + (W - W') \cdot RW$$

where W is snow moisture content (volumetric content of liquid water per unity volume of snow); wc is macimum water-holding capacity of snow (also in volumetric units).

2.2 Snow melting

Changes in snow characteristics during its melting are calculated by the following formulaes:

$$\Delta H_T = \frac{MELT \cdot RW}{DSN' - W'RW};$$

$$WM = \frac{W'H' + MELT}{H' - \Delta H_T};$$

$$W = \begin{cases} WM, & \text{at} \quad WM < WC; \\ WC, & \text{at} \quad WM \geq WC; \end{cases} \tag{8}$$

$$DSN = DSN' - W' \cdot RW + W \cdot RW$$

$$YIELD = \begin{cases} 0, & \text{at } WM < WC; \\ (WM - WC)\ (M' - \Delta M_T) & \text{at } WM \geq WC. \end{cases}$$

Here MELT is a layer of snow, melted at the Δt interval (expressed as water equivalent).

2.3 Freezing of meltwater in snow

The maximum possible amount of water, that can freeze for the Δt period (FRMAX) is calculated for the heat-exchange conditions at the snow-atmosphere interface. The actual layer of frozen water FROST and residual moisture content in snow is calculated by

$$FROST = \begin{cases} FRMAX, & \text{at} \quad FRMAX < W'H; \\ W'H, & \text{at} \quad FRMAX \geq W'H; \\ O, & \text{at} \quad W' = O; \end{cases} \qquad (9)$$

$$W = W' - FROST / H$$

2.4 Evaporation of snow

The change of snow depth during its evaporation is calculated as

$$\Delta H_E = \frac{ESN \cdot RW}{DSN} \qquad (10)$$

where ESN is a layer of snow, evaporated during Δt period (in water equivalent).

2.5 Compaction of snow

For the description of snow compaction and settling under the influence of wind loading and gravitation forces the following empirical equation is used:

$$DSN = C \; (DSN')^2 \, H' \exp \, (C2 \cdot TSNOW - C3 \cdot DSN') \, \Delta t + DSN' \qquad (11)$$

as well as the balance equations

$$\Delta H_C = \frac{DSN'H'}{DSN} \, ;$$

$$W = \frac{W'(H-H')}{H} + W' \, ; \qquad (12)$$

where TSNOW is the temperature of the snow, C a parameter, C_2 and C_3 are empirical constants.

2.6 Depth of snow

The resulting of snow cover depth at the t moment of time is determined as

$$H = H' + \Delta H_p - \Delta H_T - \Delta H_E - \Delta H_C. \tag{13}$$

3. Phase transformations of water in snow cover

Heat flux at the snow surface is calculated using the method of energy balance, suggested by P.P. Kuzmin [3]:

$$RTOT = RSN + RLN + RSEN + RLAT + RP, \tag{14}$$

where RTOT is a total flux of heat to the snow surface; RSN is a flux of short-wave solar radiation, penetrating into the snow; RLN is the effective long-wave radiation; RSEN is sensible turbulent heat flux; RLAT is a latent heat flux; RD is a heat flux with liquid precipitation.

3.1 Short-wave solar radiation

$$\begin{aligned} RSN &= RS \cdot (1 - A) \cdot CF, \\ CF &= 1 - F_1 \left[1 - (1-F)^2\right]^{1/2}, \end{aligned} \tag{15}$$

where RS is a flux of short-wave radiation, falling on the snow surface; A is albedo; CF is a coefficient of penetrating of short-wave radiation through tree crowns; F_1 is the parameter, introduced to take into account type of forest; F is the forest cover degree.

Snow cover albedo is calculated as

$$A = CA - DSN \tag{16}$$

where CA is a parameter.

3.2 Effective long-wave radiation

$$RLN = E\delta \left\{ (TATM + TKEL)^4 \left[F + (1-F)(a+b\sqrt{EATM}) \right] - (TSNOW + TKEL)^4 \right\} \qquad (17)$$

where EATM is the pressure of water vapour of air, E is emissivity in the longwave portion of the energy spectrum ($E \approx 0.99$), δ is the Stefan-Boltzman constant, TKEL = 273 K, a and b are constants.

3.3 Sensible heat transfer

$$RSEN = CS \cdot U \cdot (TATM - TSNOW) \qquad (18)$$

where U is wind speed, CS is a parameter.

3.4 Latent heat transfer

$$RLAT = CL \cdot U \cdot (EATM - ESNOW) \qquad (19)$$

where ESNOW is the saturation vapor pressure at the snow surface temperature, CL is a parameter.

3.5 Heat flux by rain

$$RP = CP \cdot PLIQ \cdot (TATM - TSNOW), \qquad (20)$$

where CP is a constant.

3.6 Snow melting and freezing of meltwater in snow

Snowpack during melting period is characterized by the following values: TSNOW = 0 °C; ESNOW = 6.11mb. Substituting these values into formulae (17) - (20) and we can determine RTOT. A layer of snow melting or potentially possible layer of frozen water in snow is determined from the relations:

$$MELT = \begin{cases} RTOT/CM, & \text{at} \quad RTOT > 0; \\ 0, & \text{at} \quad RTOT \leq 0; \end{cases}$$

$$FRMAX = \begin{cases} 0, & \text{at} \quad RTOT > 0; \\ -RTOT/CM, & \text{at} \quad RTOT \leq 0; \end{cases} \qquad (21)$$

where CM is the heat of fusion of ice.

4. Testing of the model

Calibration of parameters and testing of the model were carried out in the basin of the Tujuoja river (Finland). Its area is 21 km^2, and it is mainly forest.

4.1 Calibration of parameters

The model parameters were calibrated on the basis of observed data of water discharges and the characteristics of snow cover during 1977-1981. The Rosenbrock optimization procedure was used for the calibration of the parameters with a quality criteria:

$$R^2 = \frac{\sum(Q_r - \bar{Q}_r)^2 - \sum(Q_c - Q_r)^2}{\sum(Q_r - \bar{Q}_r)^2} \qquad (22)$$

where Q_c is the calculated value, Q_r is the observed value, $\bar{Q}_r$ is the average from observed values.

At the first stage parameters of the submodel, describing snow cover, were calibrated using measured snow cover characteristics (depth, density and water equivalent of snow). Calculation showed, that the best agreement between calculated and observed values is achieved, when parameters are calibrated over snow depth. Calibration over the values of snow density or its water equivalent has considerably worsend results.

This is assumed to be connected mainly with low accuracy of measuring snow density.

At the second stage model parameters describing soil characteristics and storage zone were calibrated by measuring water discharge in the outlet point.

At the third stage parameters of blocks describing energy balance and snow cover formation were adjusted against measured water discharge.

4.2 Testing of the model

Model parameters, received as a result of calibration process were used in a series of test calculations for the Tujuoja river basin in 1970 - 1976. Table 1 contains the values of quality criterion for calibrating and test series of calculations.

Characteristics	R^2 Calibration	R^2 Testing
Depth of snow	0.850	0.618
Density of snow	0.525	0.201
Water equivalent	0.807	0.620
Runoff	0.850	0.710

Figures 4 - 9 present the comparison of calculated and actual values.

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Figure 4.

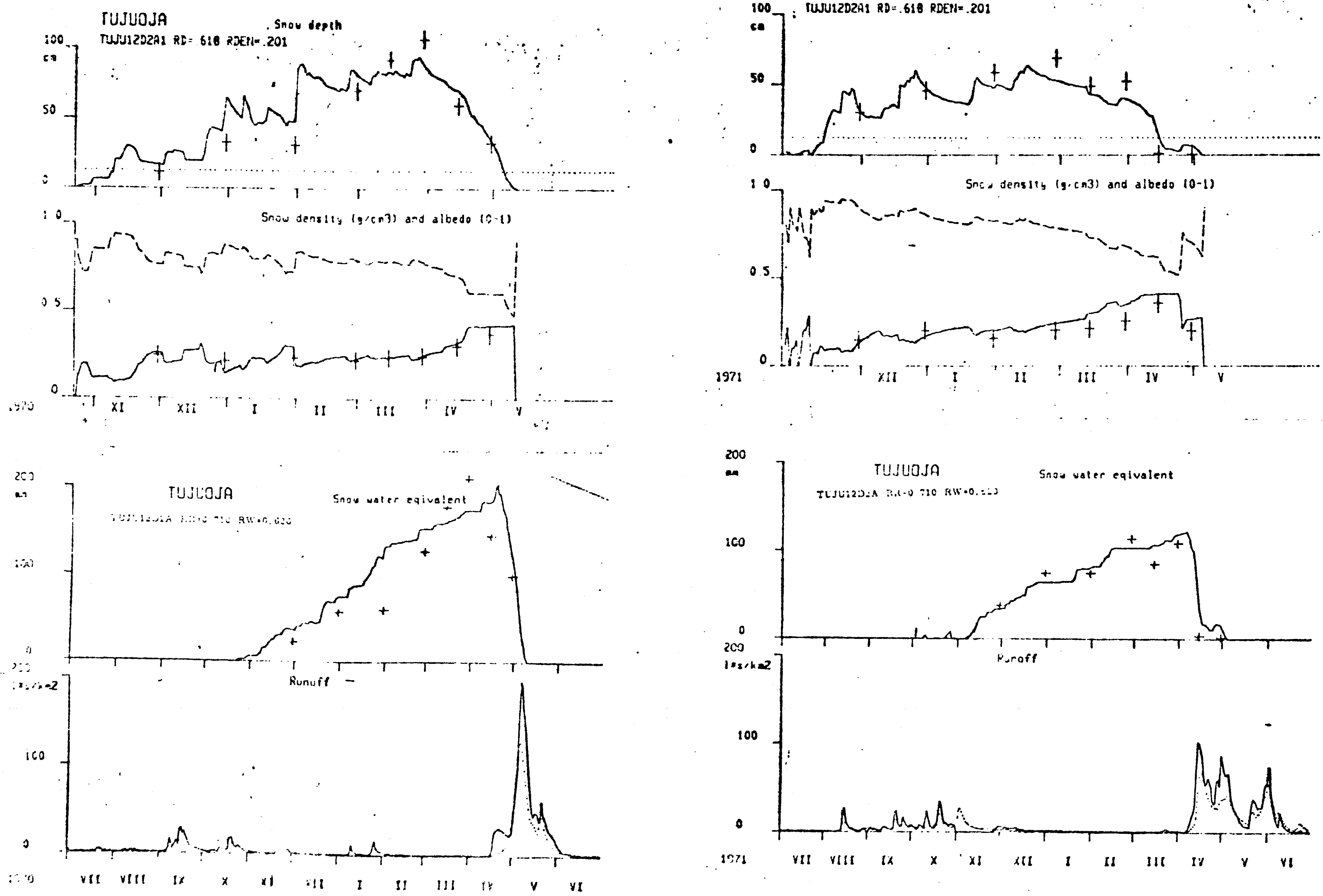

TUJUOJA
Snow depth
TUJU12D2A1 RD=.618 RDEN=.201
Snow density (g/cm3) and albedo (0-1)
Snow water equivalent
Runoff
1971
VII VIII IX X XI XII I II III IV V VI

Figure 5.

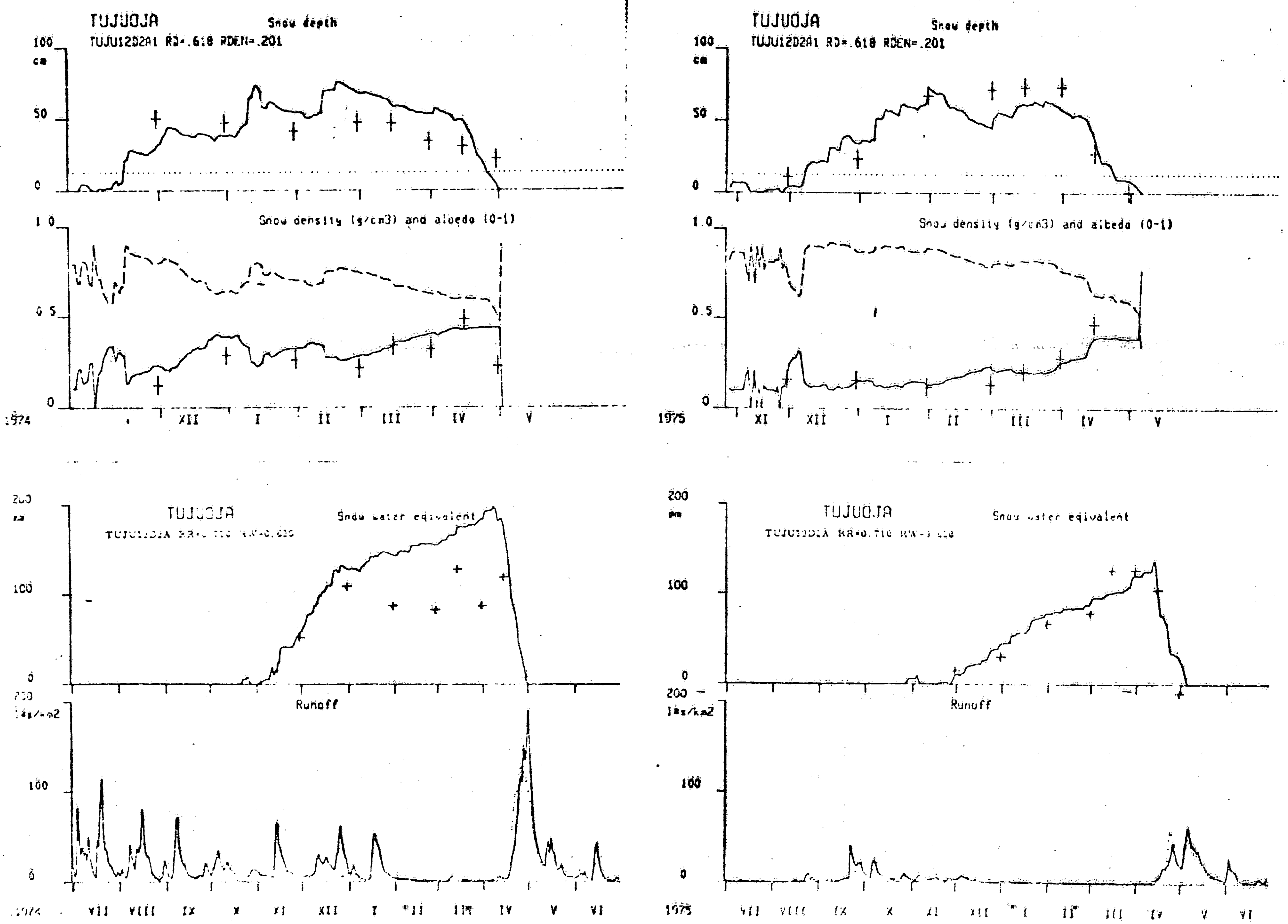

TUJUOJA
Snow depth
TUJU12D2A1 RD=.618 RDEN=.201
100
cm
50
0
Snow density (g/cm3) and albedo (0-1)
1.0
0.5
0
1974
XII
I
II
III
IV
V
1975
XI
TUJUOJA
Snow water equivalent
200
mm
100
Runoff
l/s/km2
VII
VIII
IX
X
VI

Figure 6.

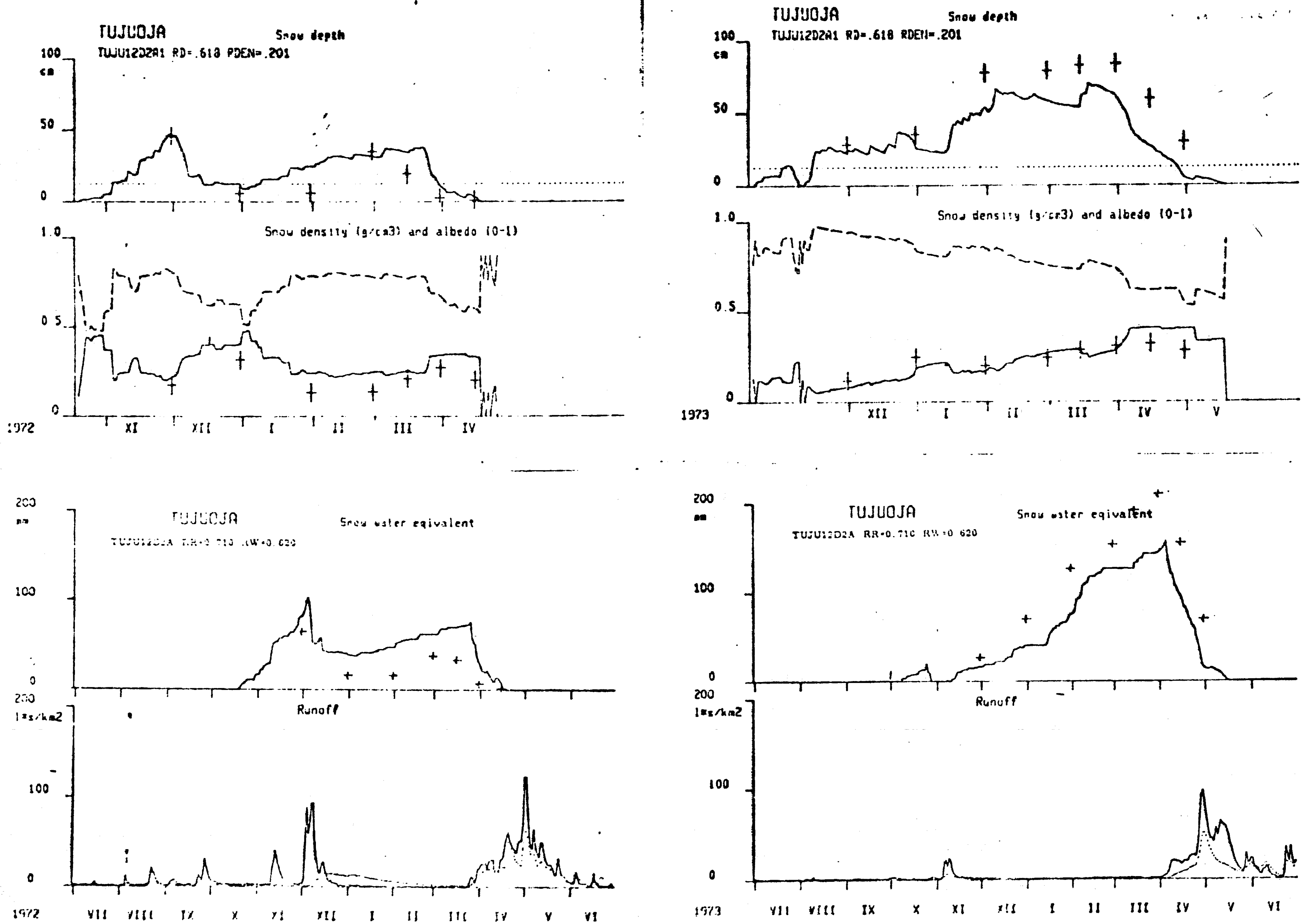
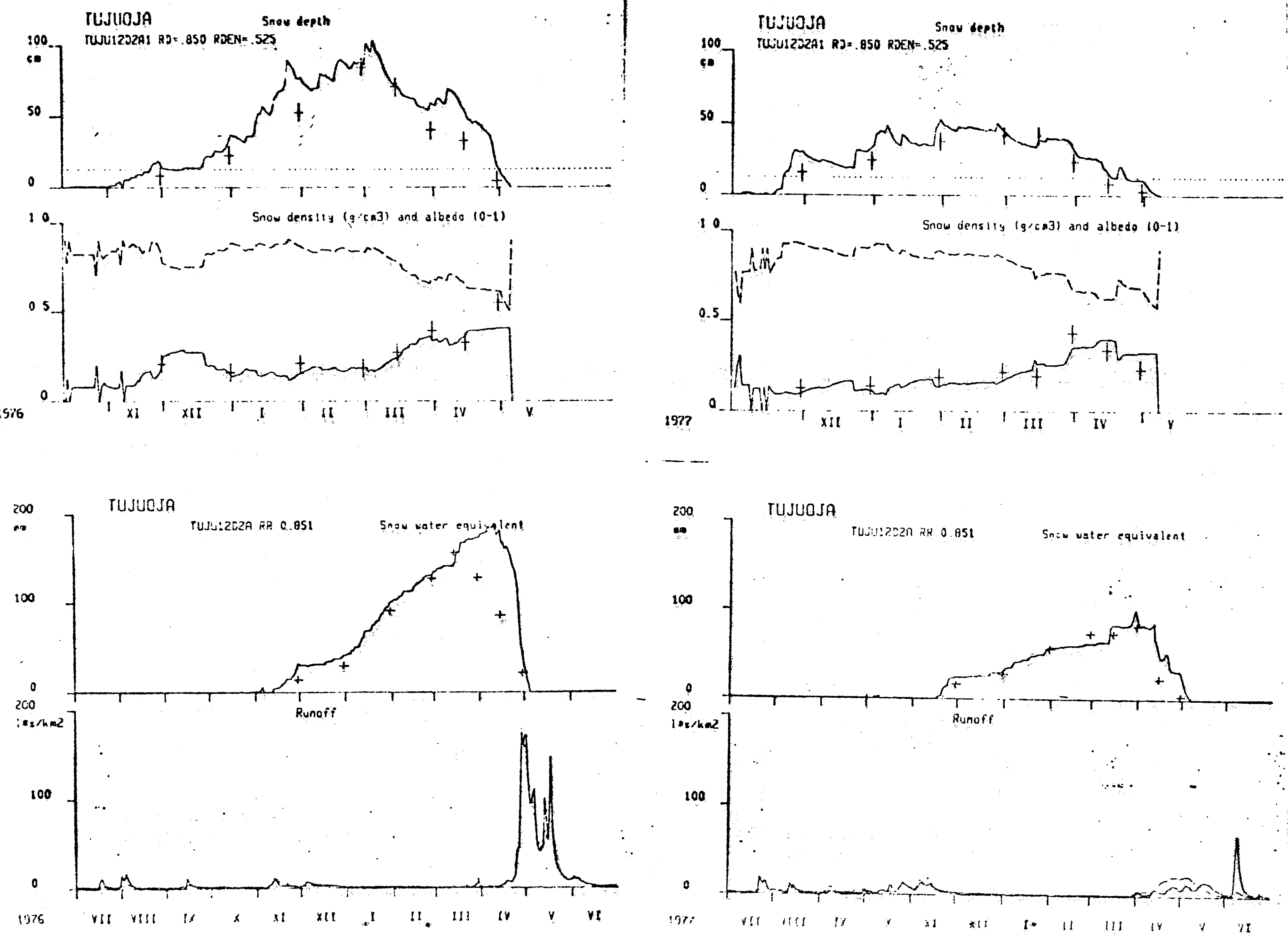

TUJUOJA
TUJU12D2A1 RD=.618 PDEN=.201
Snow depth
100 cm
50
0
Snow density (g/cm3) and albedo (0-1)
1.0
0.5
0
1972
XI XII I II III IV
TUJUOJA
Snow water eqivalent
200 mm
100
0
Runoff
200 l*s/km2
100
0
1972
VII VIII IX X XI XII I II III IV V VI
TUJUOJA
TUJU12D2A1 RD=.618 RDEN=.201
Snow depth
100 cm
50
0
Snow density (g/cm3) and albedo (0-1)
1.0
0.5
0
1973
XII I II III IV V
TUJUOJA
TUJU12D2A RR=0.710 RW=0.620
Snow water eqivalent
200 mm
100
0
Runoff
200 l*s/km2
100
0
1973
VII VIII IX X XI XII I II III IV V VI

Figure 7.



TUJUOJA
Snow depth
TUJU1202A1 RD=.850 RDEN=.525
100 cm
50
0
Snow density (g/cm3) and albedo (0-1)
1 0
0 5
0
1976
XI
XII
I
II
III
IV
V
TUJUOJA
Snow depth
TUJU1202A1 RD=.850 RDEN=.525
100 cm
50
0
Snow density (g/cm3) and albedo (0-1)
1 0
0.5
0
1977
XII
I
II
III
IV
V
TUJUOJA
TUJU1202A RR 0.851
Snow water equivalent
200 mm
100
0
200 l/s/km2
Runoff
100
0
1976
VII
VIII
IX
X
XI
XII
I
II
III
IV
V
VI
TUJUOJA
TUJU1202A RR 0.851
Snow water equivalent
200 mm
100
0
200 l/s/km2
Runoff
100
0
1977
VII
VIII
IX
X
XI
XII
I
II
III
IV
V
VI

Figure 8.

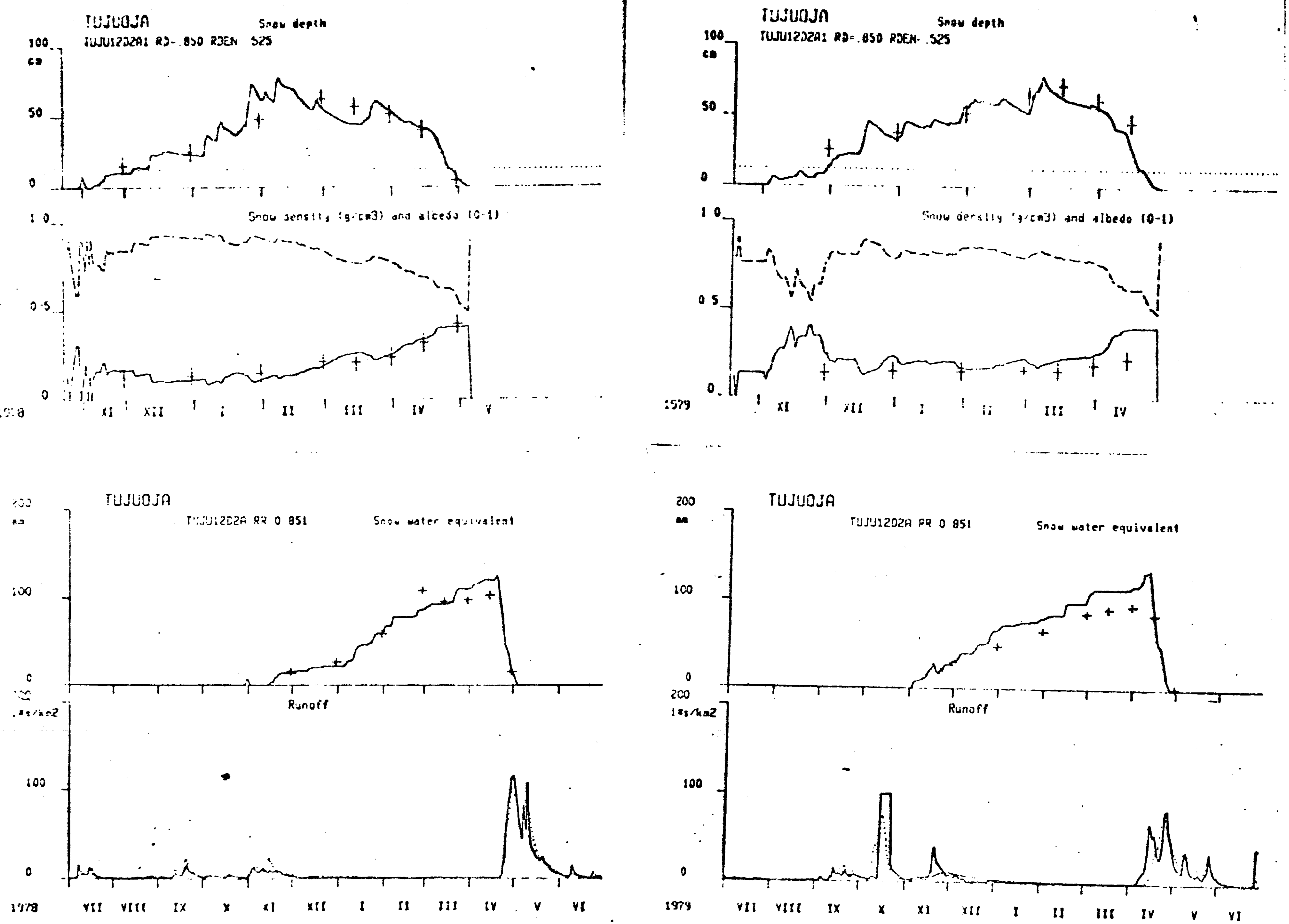
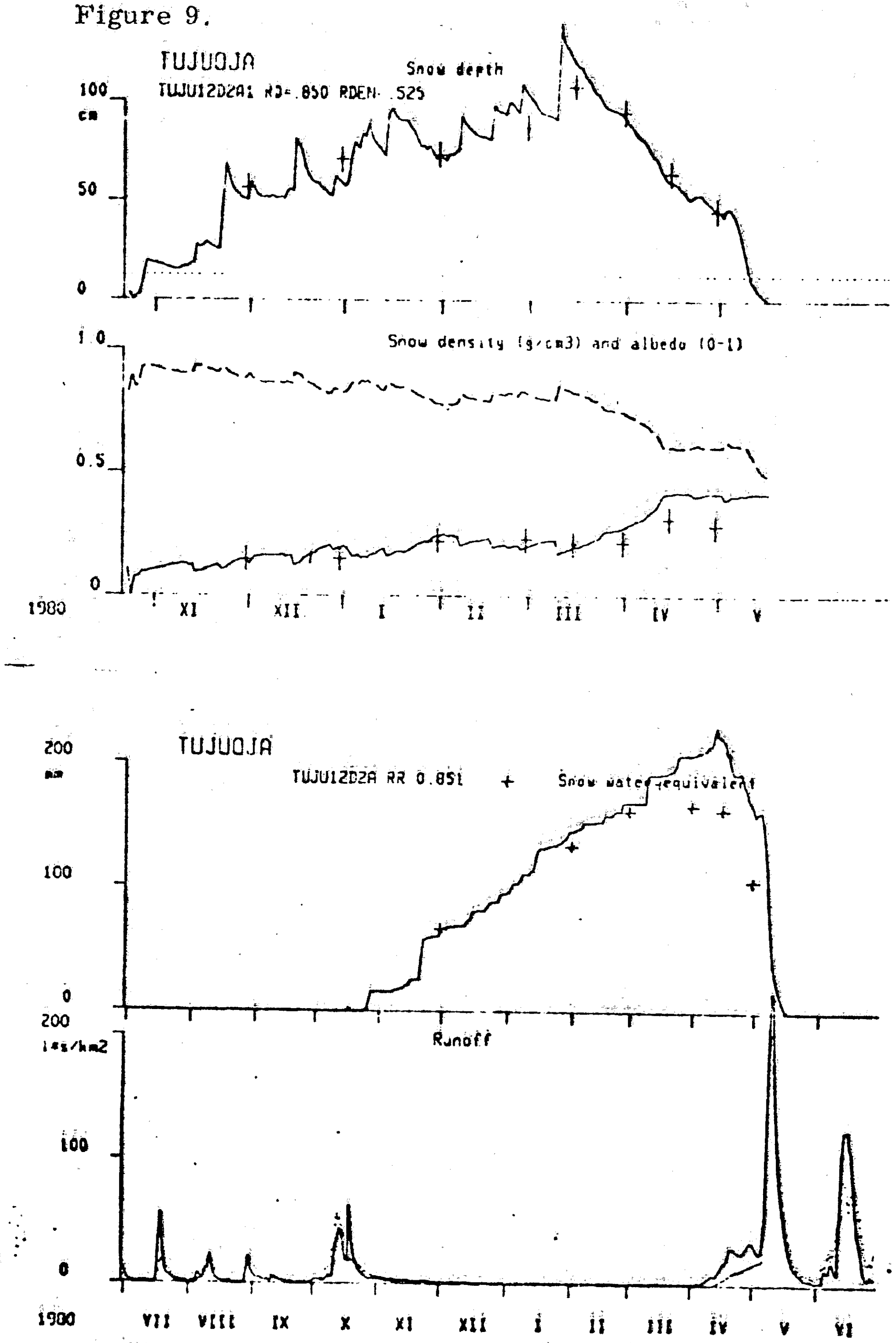

TUJUOJA
Snow depth
TUJU12D2A1 RD=.850 RDEN=.525
Snow density (g/cm3) and albedo (0-1)
1978
1979
TUJUOJA
TUJU12D2A RR 0 851
Snow water equivalent
Runoff
l/s/km2
XI XII I II III IV V
VII VIII IX X XI XII I II III IV V VI

Figure 9.



Snow depth, Snow density, Snow water equivalent

-full line is calculated
-cross marks are observed values

Runoff
-full line is observed
-dot-line is calculated

SPECIFIC FEATURES OF THE WATER DYNAMICS IN DIFFERENT TYPES OF LAKES

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J. Sarkkula and J. Koponen (Finland)

Three-dimensional mathematical models and results of propabilistic analysis of observational data have been used to study specific features of the synoptical and meso-scale currents in the lakes of different size and shape. The lakes Ladoga and Krasnoye in the USSR and Näsijärvi in Finland have been taken as targets. These lakes are located in similar physical and geographical conditions. Substantial differences in their water dynamics may occur due to different size and shape of the lake basins. For this reason the barotropic and baroclinic Kelvin and Puankare waves, governing the formation of the lake-water dynamics, can occur or cannot /1/. It is of interest to assess possible contributions from baroclinicity, bottom relief and winds, since no unique assessment has been made, so far, of the contributions of these factors to the formation of the currents in lakes. With lake Näsijärvi as an example, numerical experiments were carried out using two 3-D hydrodynamic models /3,4/. The results of calculations were intercompared and an attempt was made to verify them, based on observations of the currents made at three buoy stations (BS) on lake Näsijärvi .

Observational data from the BS test area served the basis for studies of the spectral structure of the currents and water temperature in these lakes (Fig.1).

Observations lasted for more than two months, their discreteness varying from 10 minutes to an hour; it made it possible to study the variability of the currents on meso- and synoptical scales. The spectra of the currents and water temperature for these lakes were calculated using a single technique with similar parameters /2/. In particular, taking into account that the process of the variability of the currents is vectorial, the correlative and spectral tensors of the currents were calculated with four invariant and two non-invariant characteristics. Since the process of the currents and temperature variability is nonstationary in mathematical expectance $\overline{m}$ and dispersion σ_u^2 , nonstationary quantities $K^*(\tau, t)$ and $S^*(\omega, t)$ were calculated.

An estimation of phase velocity for the baroclinic and barotropic modes and the Rossby deformation radii R = c/f gave the following results (Table 1).

Table 1 Characteristic Size of the Target Lakes and Estimates of Phase Velocity C_{ph} and the Rossby Deformation Radius R.

Lake	Size km	Average depth m	Phase velocity			
			Barotropic		Baroclinic	
			C_{ph}, m/s	R, km	C_{ph}, m/s	R, km
Ladoga	200x100	51	22	220	0.16	2-5
Näsijärvi	15x10	15	20	100	0.14	2-5
Krasnoye	7x1	10	10	100	0.04	0.5-1

According to Table 1, the barotropic Kelvin waves of mode I cannot exist in these lakes, since the characteristic size of the lakes is less than the external radius of deformation R. The internal Kelvin and Puankare waves can appear in a large stratified (in summer) lake such as Ladoga. They are possible in Näsijärvi , but in a small lake, typical of C-3 lakes (like lake Krasnoye) they are scarcely probable. Here, judging by the obtained estimates, the effect of the Earth's rotation on the water dynamics is small, since $L/R \ll 1$. In lake Näsijärvi the baroclinic Kelvin and Puankare waves must be distorted substantially by an indented coastline and uneven bottom relief.

Let us consider the spectral structure of the currents and water temperature of the lakes, based on observational data. The tensor of spectral density of the currents is determined with the use of single and reiterated Fourier transformations of the correlative tensor /2/.

Figure 2 shows the frequency-temporal spectra (linear invariants) of the currents of lakes Näsijärvi and Ladoga. It is difficult to draw the $S_u(\omega,t)$ spectrum for lake Krasnoye since the currents in this lake, driven by synoptic-scale winds, are episodic, with a life-time of several days. Then they may sease for a period of several hours to two days. As seen from Fig.2, the spectra of the currents in lakes Ladoga and Näsijärvi illustrate their non-stationarity. The spectral constituents can be selected at frequencies 0.12-0.24 and near 0.45 rad/hr. The first can be caused by synoptic-scale winds and manifest through baroclinic Kelvin waves with a frequency $\omega > f$ (f- is the local inertial frequency On a meso-scale, the spectra of the currents show motions with a frequency close to a local inertial frequency, f. These motions in the upper layers of the lake can be caused by purely inertial oscillations, excited by inhomogeneous winds in time scales less than $\tilde{\tau}<\tilde{\tau}_{in}$, and by the baroclinic Puankare waves, which exist in lakes at a distance of several radii of the inertia circle from the shore, $r > 2r_{in}$, i.e. at a distance > 3 km.

According to the probabilistic analysis of the currents, water temperature and wind speeds for these lakes, the synoptic-scale oscillations of the currents, observed at a distance of radius R in lake Ladoga, are connected with the baroclinic Kelvin waves which show themselves here even in the absence of the synoptic-scale wind fluctuations. But low-frequency oscillations exist in lakes even at a distance exceeding radius R. These motions are caused by coherent oscillations of wind speed over the lakes. The characteristic increase of the velocity of the currents in the coastal zone of the lake, confined to radius R - the so-called coastal flow /1/ - is connected with the Kelvin waves observed in a large stratified lake. Inertial motions (the Puankare waves) in lakes are of alternative character.

In lakes Ladoga and Näsijärvi their life-time does not exceed two-three inertial periods. They may be absent for about the same time period. As our observations show, these motions are lacking in lake Krasnoye. The effect of the Coriolis force on the formation of the currents is determined by the wind-speed oscillations. The time shift in maxima in the low-frequency and meso-scale spectral region shows the possibility of nonlinear mechanisms for the motions at an inertial frequency. The mutual-correlation function for the currents and water temperature for lakes Ladoga and Näsijärvi shows that the quasi-periodic constituent with inertial frequency and synoptic-scale fluctuations prevail in the spectra of the considered processes. A shift in τ by $K_{\vec{u}T}(\tau)$ illustrates that inertial motions in water temperature oscillations are delayed by several hours with respect to variations in the currents with frequency f . An analysis of data showed that in the bulk of a stratified lake at a distance of several kilometers the oscillations are manifested through the Puankare waves at frequencies $\omega \geq f$. They are generated by the shifting effects of the currents due to the inhomogeneous horizontal constituent of the velocity of the currents.

So, in lakes with a horizontal size L much larger than R, the synoptic- and meso-scale currents are determined by the Kelvin and Puankare waves. Here the nonlinear effects in the formation of specific currents in the coastal zone, confined to R, are substantial. In mathematical modeling of the dynamics of lakes it is essential to thoroughly resolve (by choosing a grid step) the coastal zone and place the emphasis on the parameterization of sub-grid processes, and, in particular, on the choice of K_Z, K_L coefficients. As calculations show /1,3/, an overestimation of K_L leads to smoothing the calculated temperature fields and loosing the water circulation (cyclonic circulation), typical of lakes, etc. The K_Z coefficient was chosen from preliminary calculations based on observations of the vertical distribution of the currents and temperature.

In the moderate-sized lake Näsijärvi the spectrum of motions is typical of that for large lakes, and data of BS observations are available for this lake that served the basis for calibrating and verifying the models. The level and the currents of this lake were calculated using a non-linear multi-layer model /3/, which has been used earlier to calculate the currents in lakes Ladoga, Sevan and Biva.

In a time scale of modelled currents the coefficients of horizontal turbulent exchange were first estimated with Hesselberg and Kolmogorov's formulae, using the spectra of the currents' fluctuations /1/. The K_L values ranged between 10^3 and 10^5 sm^2/s, differing for the coastal and open-water zones of the lakes by an order of magnitude. However, this fact has been neglected in the preliminary calculations. The most acceptable estimate of K_Z was 1-10 cm^2/s, based on observational data and preliminary calculations of the thickness of the Ekman upper layer. Results of calculations with the model from /4/ are given in /5/. Calculations were performed with a grid step of 1000 m, and the lake was vertically divided into six layers. The grid step was much less than the internal radius of the Rossby deformation, $\Delta x < R$, which made it possible to reveal the effects caused by the internal Kelvin waves. The time step was 55 s, which enabled one to determine the effects caused by the Puankare waves , in time scales equal to and less than $\tau_{\text{ин}}$ - a period of inertia for the lake width, equal to 13.5 hr. However, the spatial resolution of the grid did not permit one to describe these motions. For the upper layer the coefficient of horizontal turbulent pulse exchange was taken 8 cm^2/s and for the thermocline area 1 cm^2/s; the coefficients of horizontal momentum exchange was taken 10^4 cm^2/s. The field of currents, by models /4/ and /3/, was calculated for a period of thermal stratification (August) at a wind speed of 4 m/s, in the direction 190°. The currents were calculated with the nonlinear diagnostic model /3/ from the field of water temperature, obtained with model /5/ with a grid step Δx = 500 m on horizons 0;2.5;5;8;12.5;15;20, and 25 m. The coefficient of the vertical turbulent exchange was assumed to be constant and equal to 10 cm^2s.

Let us consider the results of calculations made with these models: we shall compare them with observations at three BS on horizons 2.5 and 8 m, made during the same period.

Figure 4 shows the results of calculations of the lake level ζ using models /3/ and /4/. Deviations of the level from the undisturbed state vary between +30 and -20 mm (model /5/), and between +13 and -10 mm (model /3/), i.e. according to model /3/, the velocity must be somewhat less, and the topography of a free surface ζ does not differ qualitatively in these models. Table 2 gives the results of statistical analysis of the field of velocities calculated with models /3,4/ for horizons 2.5 and 8 m. The velocity was averaged over a field of 75 points for horizon 2.5 m and of 61 points for horizon 8 m. The results show that the average velocity of the currents from model /4/ is higher than from the diagnostic model; average (by module) velocities of the currents from model /3/ are 3.3 cm/s, and from model /4/ 4.7 cm/s; for horizon 8 m, average (by module) and maximum velocities of the currents differ slightly. An analysis of the fields of the currents (Fig.5) from models /4/ and /3/ shows that in the upper 0-5 m layer the wind-driven currents prevail, and beneath this layer the effects of baroclinicity and bottom relief contribute most.

Calculations of the modelled field of currents were compared with observations made at the BS for horizons 2.5 and 8 m. Data of the 10-min discrete observations were averaged over 30 hours, which corresponded to the time period needed for the currents to form, according to model /4/. Table 3 lists the results of this comparison. The direction of the currents was assessed to an accuracy of ±20°, and the velocity of the currents ±1 cm/s. Since the BS co-ordinates did not coincide with the grid knots, the direction and velocity of the currents were averaged over the values at 3-4 points of the grid area. As seen from Table 3, in the 0-2 m layer the observed velocity of the currents is higher than the modelled one. The results of measurements in the upper layer show that the currents flow mainly at an angle of 20-40° with respect to wind direction, the variability of the currents in the upper layer is slow and no inertial and other oscillations were observed here in this period. For horizon 8 m, both the observed and modelled velocities and directions of the currents coincide.

Conclusion

Studies have been carried out of the currents in lakes of different size and shape; the transformation of the spectra of the currents, depending on the hydrometeorological conditions as well as the shape and size of lakes, has been shown.

The currents in lake Näsijärvi were calculated for a period of stratification using nonlinear diagnostic and prognostic models. Comparisons have shown that, qualitatively, the results of calculations of the fields of velocity are not controversial and agree reasonably well with the 30-hr averaged data of measurements at the BS. For a more correct verification of the models, more BS are needed, which will take into account specific circulations of the lake water. As shown by calculations of the currents with models /3,4/, the BS should total not less than 7.

Calculations have also shown that in the upper 0-5 m layer the wind effects prevail, and beneath it - the combined effects of baroclinicity and bottom relief are observed.

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5. M.Virtanen, j.Koponen,K.Dahlbo,J.Sarkkula. Three dimensional water quality transport model compared with field observations. Ecological Modelling, 31(1986) 185-199.

Table 2

Average componens of vector currents, on horizons 2 and 8 m in Näsijärvi

Horisonts m	Parametr	Model Simons/4/	Model Sarcisjn /3/
2	$\bar{u}$	2,5	o,2
	$\bar{v}$	3,4	o,7
	$\lvert V\rvert$	4,7	3,3
	V_{max}	16	9
8	$\bar{u}$	-o,5	- o,9
	$\bar{v}$	2,8	o,2
	$\lvert V\rvert$	2,5	1,7
	V_{max}	8	7

Table 3

The **results** of simulations of currents on models /3,4/ and experimental data on BS. Period of **averaging** 3o h

Horizon	BS number	Predict. model /4/ dir.	Predict. model /4/ speed	Diagnos. model /3/ dir.	Diagnos. model /3/ speed	Experimental field data dir.	Experimental field data speed
2	1	3o	8	1oo	5	5o	16
	2	4o	8	3oo	3	3oo	8
	3	5o	9	19o	8	4o	16
8	1	19o	5	19o	3	14o	5
	2	27o	5	28o	3	19o	3
	3	28o	4	19o	4	33o	4

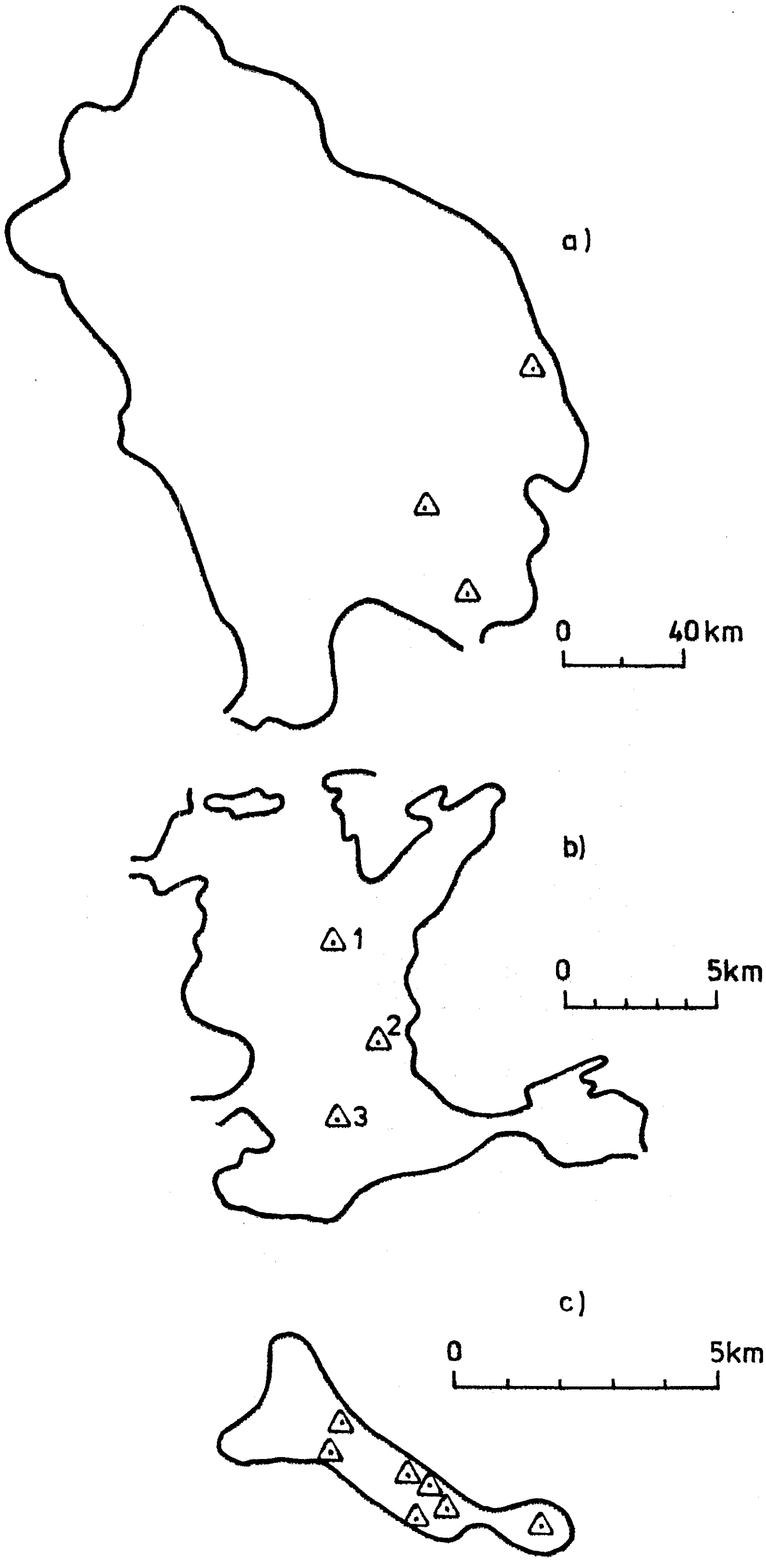


Fig. 1 Distribution of buoy stations in lakes Ladoga (a), Näsijärvi (b), Red-Lake (c)

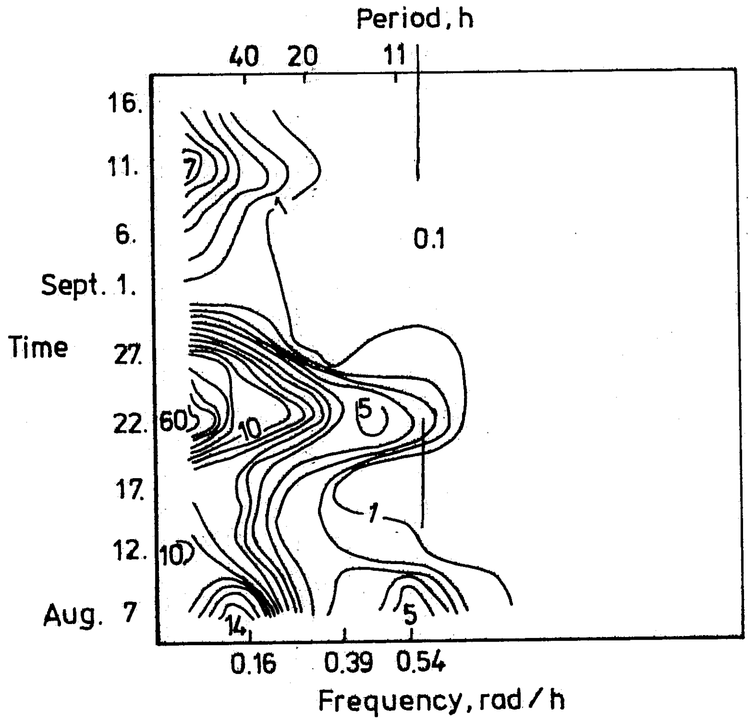


Fig. 2 Non-stationary current spectra of Lake Näsijärvi

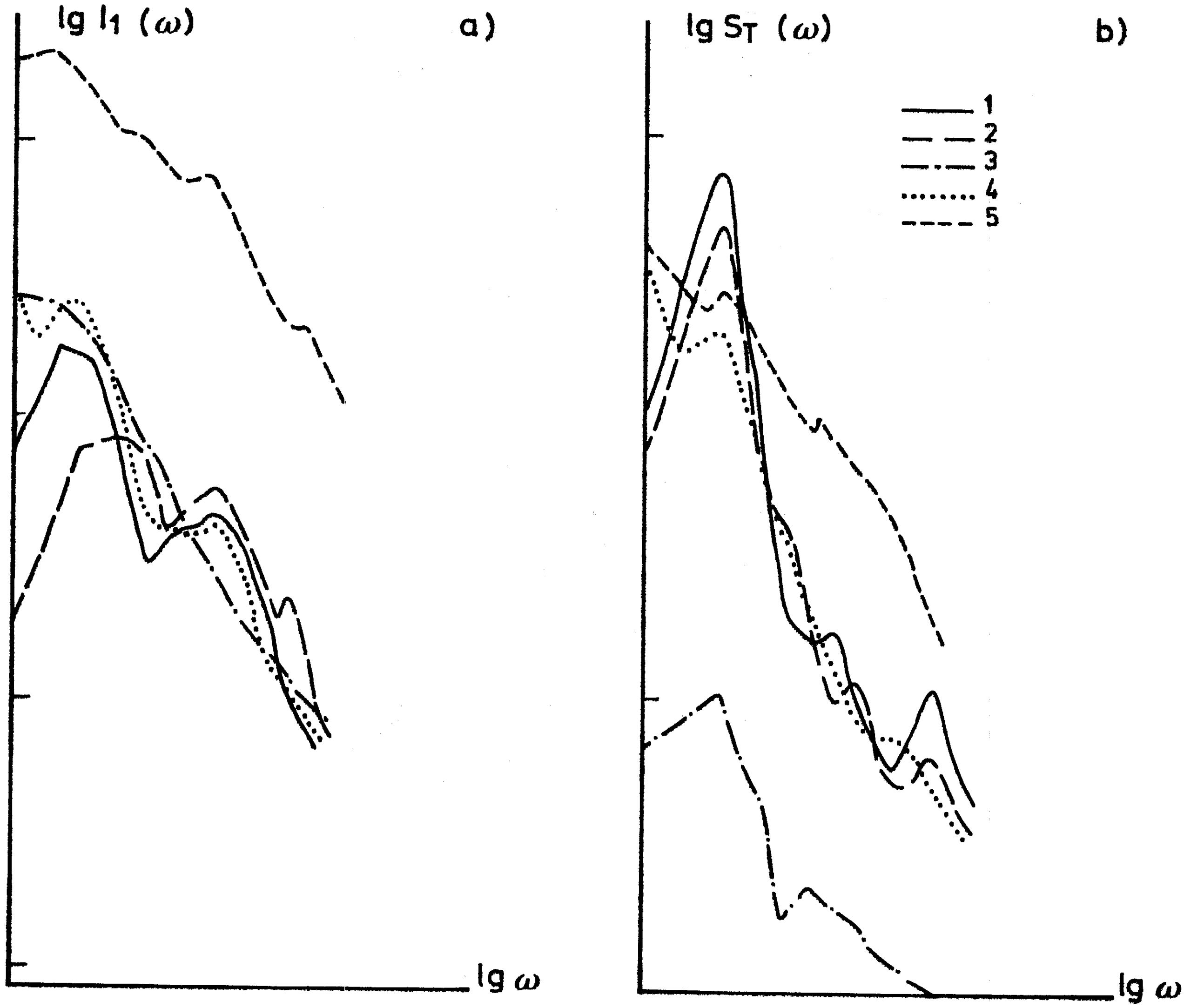


Fig. 3 Spectra of currents (a) and water temperature (b) of Lake Näsijärvi (1,2 and3), Red-Lake (4), Ladoga (5)

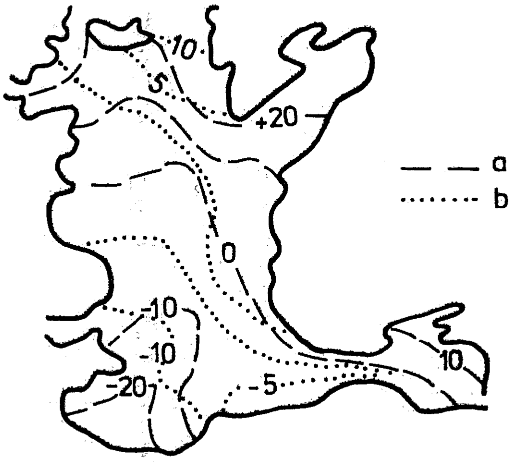
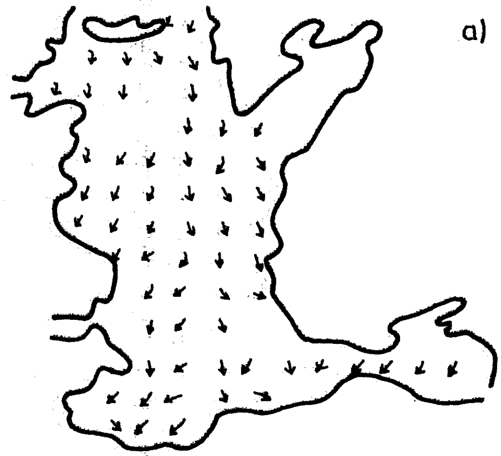


Fig.4 Level of free surface of Lake Näsijärvi
a) multilayer model /4/
b) diagnostic model /3/



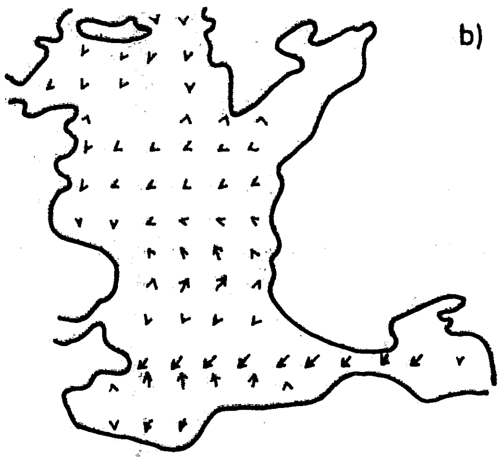


Fig.5 Currents of Lake Näsijärvi. Horizon 8 m.
a) Simons model /4/
b) Sarkisjn model /3/

TRANSFORMATION OF NATURAL ORGANIC MATTER IN WATER STORAGE RESERVOIRS

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GREKOV V.F., LAPIN I.A., Hydrochemical Institute

River flow controlling has a key effect on component composition of the water which is manifested in varied averaging of the river water chemical composition, its redistribution in time, changes in the velocity and duration of transformation processes involving organic and organic-inorganic compounds allochthonous in origin, as well as intensification of processes concerning synthesis of autochthonous organic matter. The paper considers basic factors combination of which governs transformation of natural organic matter in water storage reservoirs.

Considered as an important factor of organic matter transformation in a water reservoir is the intensity of water circulation conditioning duration of the effect exercised by physico-chemical and biological processes on autochthonous and allochthonous organic matter. Present day theoretical approaches in describing the reservoir velocity field based on combined integration of equations in terms of hydrodynamics and physico-chemical kinetics are rather difficult to apply, for the complicated geometry of boundary conditions characteristic of tributaries does not allow correct solutions.

The technique elaborated at the Water Institute of the USSR Academy of Sciences is based on substituting the analysis of particles' paths with that of the time of their residence in the water storage reservoir. With this approach the function of residence time distribution (RTD function) is a key factor in describing the water circulation intensity and pattern. For flowing water reservoirs the RTD function is rather clear in meaning: its value at a given moment t is the probability that a flow element entering the reservoir at zero time t_o would stay in it during a period shorter than the said t-time. Substance transformation processes in a given reservoir develop in function of time and different flow elements reside in the reservoir during varied periods of time which means that the RTD function if known allows the assessment of the level the transformation process of a given substance has reached.

The RTD function is equivalent to an impulse transient function of the system. For the case of water reservoirs the impulse transient function is represented by a curve of variation in substance concentration at the outlet, the substance inflow at the inlet being of volley type. In such a dynamic system as a reservoir it is practically impossible to determine the impulse transient function directly. So the RTD functions are experimentally obtained from hydraulic investigations employing water reservoir models.

Theoretically the hydraulic modelling is grounded on the Navier-Stokes and continuity equations. Investigation data attest to a possibility of using forced flows, provided both geometrical and dynamical similarity could be achieved. When the data inferred from model investigations are extended to real-life objects, the indices of inner water circulation intensity in reservoirs are not notably affected by the employed forced flows and model distortions.

Model experiments serve to yield data on the curves of a hydrodynamic system response to a pulse injection of chemical indicator. Besides that, hydraulic experiments provide ground for obtaining an idealized hydrodynamic pattern of water ciculation in the given reservoir. The hydrodynamic structure is divided into ultimate blocks and units regarded as simple in respect to methematical description of the hydrochemical processes that are under way. Serving as such units are thoroughly studied hydrodynamic structures of mixer type. This division results in substituting the actual hydrodynamic structure with an equivalent schematic description calculated from equations of material balance. If each of the units has a corresponding differential equation, the combination of differential equations for all the units coupled with algebraic equations of material balance for nodal points of the block scheme constitutes a mathematical model of the whole water storage reservoir. Individual schemes of water circulation require also individual structure of mathematical description concerning the reservoir dynamic characteristics that is RTD function. With multi-unit ramified schemes of water circulation the reservoir dynamic characteristics result rather complicated in form and difficult to make use of in calculations. With the aforesaid in view it is reasonable to substitute the individual water circulation scheme by a dynamically equivalent typical structure of logitudinal-transversal water circulation with the parameters providing the required RTD function at the reservoir outlet. The parameters of this mathematical model are computer-selected following the criterion of minimal deviation of the model RTD functions from experimental ones.

The abovesaid may be illustrated with the data obtained through studying water circulation processes and organic matter transformation at the Uchinsk water storage reservoir of $146 \cdot 10^6$ cu.m utilized for water supply of Moscow. The Uchinsk water storage reservoir was simulated after the Froude criterion.

Scale relationship is of the form:

$$\alpha_v = \frac{\alpha_h}{\alpha_e^{0.5} \cdot m}; \quad \alpha_g = \frac{\alpha_e^{0.5} \cdot \alpha_h^2}{m}; \quad \alpha_t = \frac{\alpha_e^{1.5} \cdot m}{\alpha_h}$$

where $\alpha_v, \alpha_g, \alpha_t, \alpha_e, \alpha_h$ are scale coefficients of velocity, flow, time and vertical and linear dimensions respectively, m is the degree of flow forcing in the model. The model was run with a nine-fold distortion of scale ($\alpha_e = 900$, $\alpha_h = 100$).

Curves of the water reservoir response to an indicator injection were inferred from the results of ten fluoresceine injections with a stationary functioning of the model. Scattering of the experimental RTD functions proved to be insignificant and the yielded data could be averaged (Fig.1).

Model experiments allowed the identification of main flow patterns (Fig.2a). These experiments provided a sound basis for compiling a block schemes of water circulation (Fig.2b) substituted with a dynamically equivalent unified one with the parameters providing the same value of RTD function (Fig.1b). Parametrical identification of the reservoir mathematical model employs this unified structure of longitudinal-transversal water circulation. Each cell represents a mixer. The model parameters of each tract are as follows: net lag time τ, number of unified two-cell blocks n, time constant of a unified block T, relative volume of slow water zone X, relative intensity of water circulation between the transit and slow water zones y. These parameters were computer-selected through reiterative search following the criterion of maximal approximation of computer model response to δ-functional impact to the experimental RTD function. Deviation of the final solutions from the hydraulic model regressive RTD functions does not exceed 10% in the maximum range and 20% in the tails of experimental curves for both tracts. Final parameters of the simulated water exchange in the Uchinsk water storage reservoir are as follows:

For the tract "Pestovo - NWSS"

$\tau_1 = 4.0$ days, $n_1 = 5$, $T_1 = 7.4$ days, $X_1 = 0.55$,

$y_1 = 0.30$;

For the tract "Pestovo - EWSS"

τ_2 = 13.1 days, n_2 = 7, T_2 = 6.9 days, x_2 = 0.55, y_2 = 0.35

It should be noted that net lags of the mathematical model are appreciably less than those descriptive of the response curves. This is assigned to the fact that the mathematical model summed lags which should coincide with the last ones of the response curves, include both the said τ_1 and τ_2 and those emerging in the integration of longitudinal-transversal water circulation equations. With n = 6 - 7 and the total residence time of many days, last lags are considerable and amount to 2 days for the first tract and to about 5 days for the second one. Summed up with τ_1 and τ_2 they amount to 6-day travel time lags at the NWSS and to 18-day lags at the EWSS.

Retrospective checking and refinement of the water circulation mathematical model was carried out on the bases of field observation data obtained at the reservoir inlet and outlet. Serving as statistical data were weekly measurements of chloride concentrations over a period of 1973-1979. Chloride content variations reveal a well pronounced seasonal cyclicity in conformity with general changes in mineralization due to varied types of intake throughout the year. This makes it possible to carry out the dymanic investigations for one oscillation frequency C_{inlet} (ω = = 0.017 day^{-1}).

Over a period of 1973-1979 the NWSS and EWSS capacities were relatively stable: Q_1= 11.1 m^3/ sec and Q_2 =

$= 15.1\ m^3/sec$. In accordance with the developed model of water circulation the water reservoir time constants should be equal to the following values:

$\tau_1 = 5.4$ days, $T_1 = 10.0$ days, $\tau_2 = 13.0$ days, $T_2 = 6.9$ days.

Now we can calculate amplitude-phase transformation of seasonal variations in the non-conservative admixture concentration that should be observed over the period if the mathematical model has been built correctly. In conformity with the mathematical model structure, transformation of cyclic variations in the conservative admixture concentration by each reservoir tract is of the form:

$$A(\omega)=\left\{\frac{\left(\frac{\alpha}{\beta}T\omega\right)^2+1}{\left[1-\frac{\alpha(1-\alpha)}{\beta}T^2\omega^2\right]^2+\left[\left(1+\frac{\alpha}{\beta}\right)T\cdot\omega\right]^2}\right\}^{\frac{n}{2}}$$

$$\varphi(\omega)=n\left[\operatorname{arctg}\frac{\alpha}{\beta}T\omega-\operatorname{arctg}\frac{\left(1+\frac{\alpha}{\beta}\right)T\cdot\omega}{1-\frac{\alpha(1-\alpha)}{\beta}T^2\omega^2}\right]-\omega\tau$$

Substitution of the parameters n, α, β and also τ, T gives a transformation model $C_{inlet}-C_{outlet}$, corresponding to $Q_1 = 11.1\ m^3/sec$ and $Q_2 = 15.1\ m^3/sec$. Substituting $\omega = 0.017\ day^{-1}$ into the model we pass on to quantitative evaluation of amplitude supression and phase shift defining yearly variations in the conservative admixture concentration cited in the first line of Table 1. The second line gives amplitude supression and phase shift values of yearly variations in chloride concentration obtained from field data with the use of the first harmonic of expansion into a Fourier series of the autocorrelation function $R_{C_{inlet}}(0)$ and the intercorrelation function $R_{C_{inlet}-C_{outlet}}(0)$.

TABLE 1. Retrospective checking of the water circulation mathematical model for smoothing down the year cycle of chloride concentration

	"Pestovo-NWSS"		"Pestovo-EWSS"	
	amplitude supression	phase shift	amplitude supression	phase shift
According to the reservoir mathematical model with $Q_1 = 11.1$ and $Q_2 = 15.1$	0.825	50.8	0.881	58.7
According to field observation data over 1973-1979	0.847	45.2	0.814	52.7

The comparison of actual amplitude-phase transformations with those calculated by means of a mathematical model suggests that the water circulation model based on the data obtained from hydraulic experiments has proved to be valid. In order to fit fully the field observation data the model has been refined following the solution of error equations for A and φ. The final version of water circulation model for the Uchinsk water storage reservoir is of the form:

$\tau_1 = 53/Q_1$, $T_1 = 99/Q_1$, $n_1 = 5$, $\alpha_1 = 0.55$, $\beta_1 = 0.27$

$\tau_2 = 190/Q$, $T_2 = 101/Q$, $n_2 = 7$, $\alpha_2 = 0.55$, $\beta_2 = 0.14$

The model may be employed for calculating water circulation and the degree of conservative matter averaging in the water storage reservoir with any capacity ration of the Eastern and Northern Water Supply Stations (EWSS and NWSS) ranging from 10 to 20 m^3/day. Models of this type are basic for simulating transformation processes involving non-conservative admixtures.

One of the main factors indicating water quality of this source of water supply which conditions the pattern of water treatment at NWSS and EWSS is the water color. It is common knowledge that the main contribution to the natural water color is made by humic matters composed of mixed humic acids (HA) and fulvic acids (FA). From the available data on many-year observations of water color at the Ivan'kovsky water storage reservoir, as well as HA an FA content we have formulated empirical dependences of color vs HA concentration for the given reservoir in function of different hydraulic phases.

$C_{HA} = 0.955 - 0.0023A$ for the summer-autumn period,

$C_{HA} = 0.0004\ A^{1.533}$ for the winter-spring period.

where C_{HA} is the concentration of HA in mg C/l, A is the color index in degrees of the platinum-cobalt scale.

To complete the data on the content and composition of the dissolved organic matter, the FA concentrations may be inferred from the ratio C_{HA}/C_{FA} defined over different periods of time and equal to 0.4 in summer-autumn and to 0.12 in winter-spring.

For the water circulation conditions described by the admitted model and the mathematically postulated concentrations of color index at the inlet mC_{inlet} and outlet mC_{outlet} we have:

$$\frac{mC_{вых}}{mC_{вх}} = \frac{\left(\frac{\alpha}{\beta}TK+1\right)^n \cdot e^{-K\tau}}{\left[\frac{\alpha(1-\alpha)}{\beta}T^2K^2+\left(1+\frac{\alpha}{\beta}\right)T\cdot K+1\right]^n}$$

where K is a seasonal constant of the reservoir autopurification velocity expressed in day^{-1}.

To gain a better understanding of autopurification processes towards the elimination of naturally dissolved organic matter (HA and FA) in water bodies, there were carried out investigations of these matters' sedimentation resulting from changes in the reservoir hydrochemical conditions. The results yielded by the study of HA phase-dispersion composition under varied environmental conditions with the use of exclusion and ion-change chromatography, ultrafiltration and ultra-violet spectroscopy, suggest that the sedimantation pattern is of the following type. The HA sedimentation process in water medium is conditioned by the quantitative and qualitative composition of inorganic components and develops in a step-wise manner, On the whole the HA sedimentation is a combination of successive competitive processes. Cations of alkali-earth metals interact with low-molecular fractions (LMF) of the HA and partially neutralize the surface charge of macromolecules. Reduction of the repulsive force between macromolecules leads to the formation of larger aggregates, that is high-molecular fractions (HMF). Later on a similar interaction of Ca^{2+} and Mg^{2+} cations and HMF results in enlarged HA aggregates up to the size of colloidal fraction (CF) and further to suspended fraction, which means the formation of solid phase (SP).

On the basis of the described HA sedimantation mechanism affected by inorganic components of the solution, a mathematical model has been developed. It consists of a system of ordinary differential equations of the first order:

$$\frac{d[S_1]}{d\tau} = \beta_1 [LMF]\cdot[S]$$

$$\frac{d[S_2]}{d\tau} = \beta_2 [HMF]\cdot[S]$$

$$\frac{d[S_3]}{d\tau} = \beta_3 [CF]\cdot[S]$$

where S is the resultant concentration of inorganic components; S_1, S_2 and S_3 are the concentrations of inorganic components in each of the described processes; τ is the time.

Current concentrations of the HA fractions in the interraction process are inferred from:

$$[LMF] = [LMF_o] - 0.20\,[S_1]$$

$$[HMF] = [HMF_o] - 0.10\,[S_1] - 0.0476\,[S_2]$$

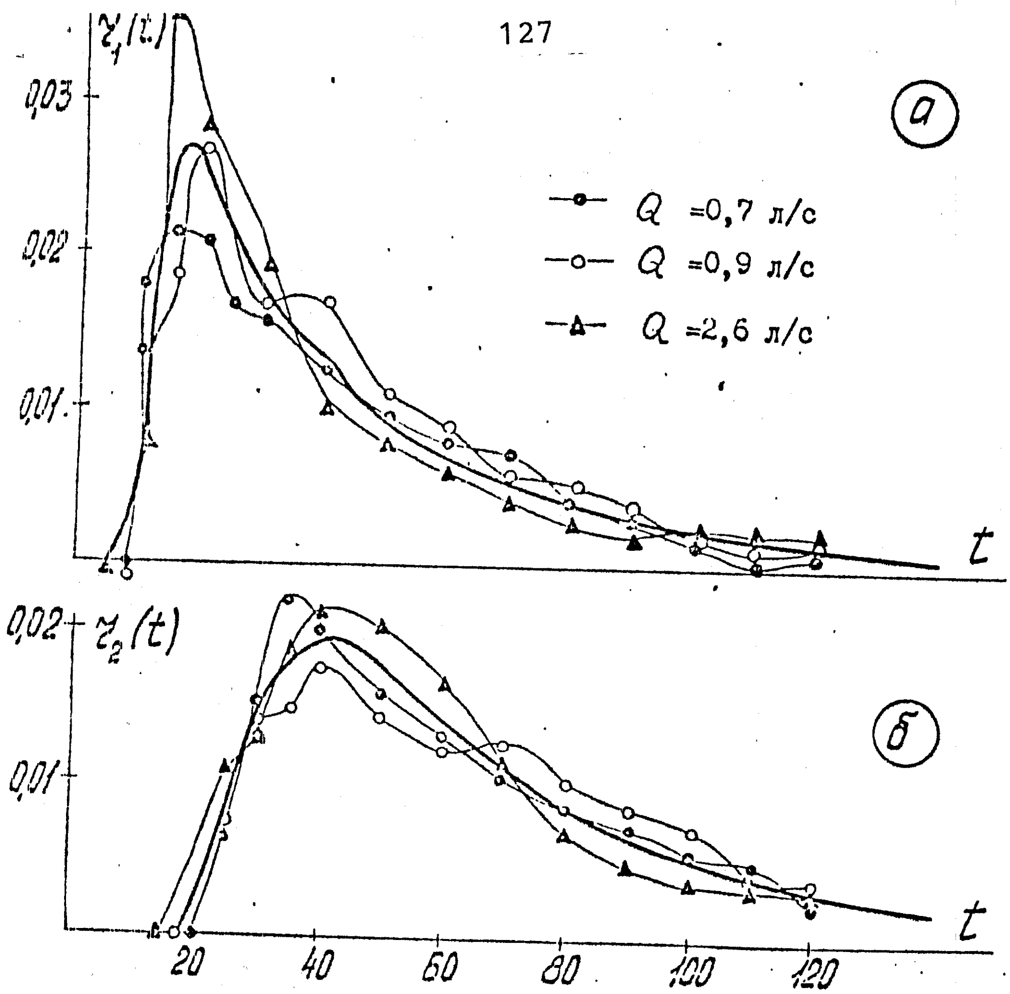
$$[CF] = 0.0119\,[S_2] - 1\cdot10^{-4}\,[S_3]$$

$$[SF] = 6.25\cdot10^{-6}\,[S_3]$$

$$[S] = [S_o] - [S_1] - [S_2] - [S_3]$$

The values of initial concentrations are indicated with the "o" index.

The suggested mathematical model allows the identification of the main directions in the process of different HA forms' emergence, helps to recognize types of interconnection between them and determine relative kinetic parameters characteristic of the said processes and thus take into account and assess quantitatively the velocity of HA sedimentation in natural waters.



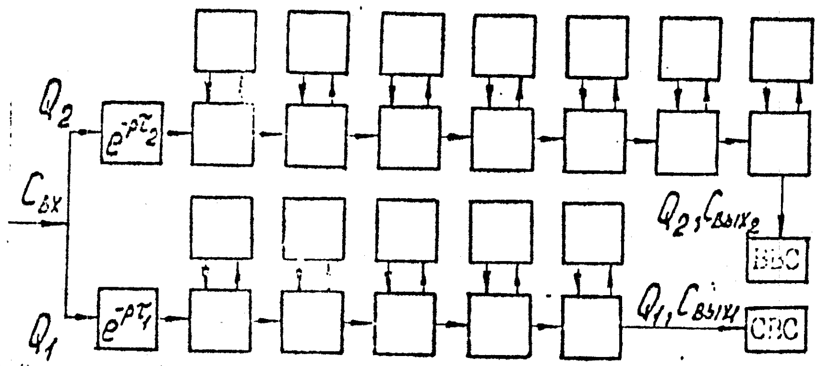


Fig. 1 Experimental RTD functions of the hydraulic model describing the Uchinsk water storage reservoir along the tracts "Pestovo - NWSS" (a) and "Pestovo - EWSS" (b) and the model of water circulation based on the unified structures of longitudinal-transversal water circulation

1 - Q = 0.7 l/sec

2 - Volga

3 - Water reservoirs of the Moscow channel

4 - Q_2, C_{outlet_2} - EWSS; Q_1, C_{outlet_1} - NWSS

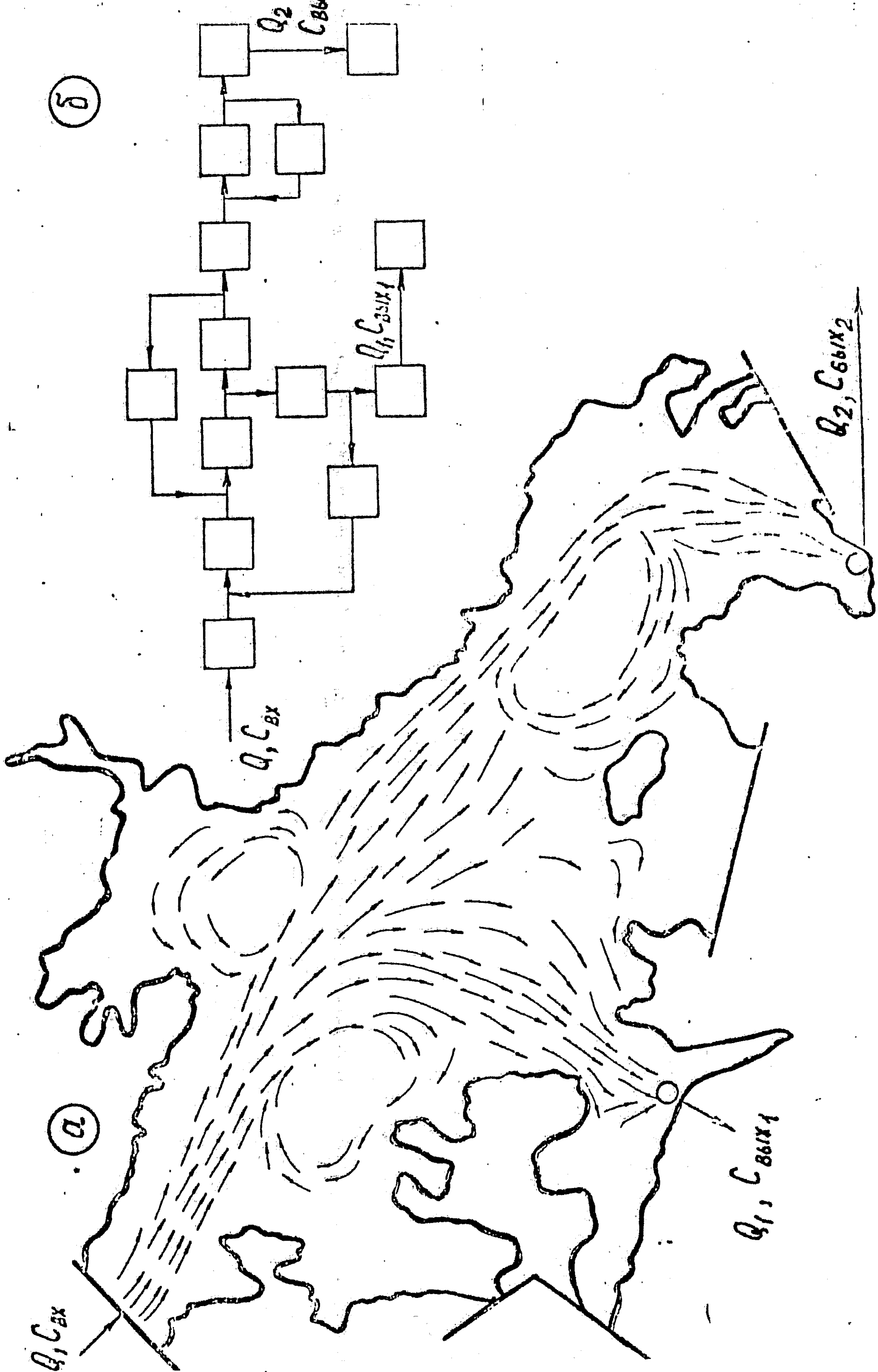


Fig.2 Scheme of the main currents in the Uchinsk water storage reservoir after experiments with the use of a hydraulic model (a) and the corresponding generalized scheme of water circulation (b)

1 -C_{inlet}

2 - C_{outlet}

WATER DYNAMICS AND MEASURING HYDROLOGICAL PARAMETERS IN CONNECTION TO THE WATER QUALITY PROBLEMS IN LAKES AND COASTAL AREAS

Timo Huttula

INTRODUCTION

During the last two decades, the importance of hydrological factors in water quality and hydrobilogical studies has been realized (Kuusisto 1984). Considerable progress has been made in the study of lake hydrodynamics. The possibilities of measuring currents have improved since the registrating meters have become reliable and widely used. Also numerical models even in 3 dimensions describe circulation and transport fairly well.

The hydrodynamic processes occuring in lakes and coastal areas can be divided into the following groups:

1. Surface phenomena
   a. wind generated waves and currents
   b. wind generated surface oscillations
   c. tidal currents
   d. waves due to changes in discharge

2. Internal currents
   a. flow through the area
   b. currents due to surface movements

3. Transport phenomena
   a. vertical mixing and stratification
   b. horizontal diffusion

All these processes have their typical spatial and temporal scales which are controlled by the scales of atmospheric phenomena and by the dimensions and geometry of the water body. This heterogenuity leads to incomplete mixing and uneven ditribution of chemical and biological constituents within the water body. Therefore simple mass balance relationhips between input and output of these constituents and their concentrations in water samples are usually not valid. This means that measurements and modelling of hydrological factors in connection to water quality problems are essential.

In planning the measuring program the following variables should be considered:wind velocity and direction, input and output discharges and loads, water level, thermal structure, water currents, sedimentation, concentration of water quality parametrs. In each particular case the decisions about what to include to the program and how and where to measure depends on the following things:

-the goal of the work/ What are the questions to be answered?
-the effect of the variable/ What order of magnitude will its effect be?
-the time available
-the money available

In the following three examples are given. With them I want to show how we have done hydrological investigation in order to help to solve water quality problems. Another example about the water quality and hydrological investigation is the investigation made in the Valkeakoski water course, where the interraction between the water currents and the erosion and the transport of celluloose fibers was studied (Huttula & Krogerus 1986).

CASE 1. STRÖÖMI

The Ströömi sound is situated in Kustavi archipelago at the south western coust of Finland. It is 20 km long with a width of 0.5 - 2 km. It is a area of very active fish farming. Fish farming has been growing very rapidly during the last decade. Farming of rainbow trout in sea was started in Kustavi archipelago in 1970 with a pruduction of 15 tons/y (Isotalo & al. 1984). Now this area is producing 3400 tons/y, which is one third of the whole rainbow trout production of the country. Consequently the enviromental effects of the farming have been under an intensive discussion. Among the questions raised were:

-how much water is going through the Ströömi sound?
-how much of the fish food is settled and how much transported ?
-is resuspension possible in Srööni ?
-what is the load of the fishfarming to the surrounding waters ?

From the bottom fauna studies and also from the interviews of fishers it was learned that water exchange in the sound is effective from time to time. The measurement program was planned for intensive growing season in the end of the summer (Fig. 1). The program included:

1. Current measurements in the sounds connecting Strööni to the sea. The meters were placed in surface and in bottom layers because a two layer flow was expected. In the middle of Strööni one meter was placed in order to check the intercorrelation of currents there and in the sounds. This meter was placed close the bottom because of the need to study the resuspension. Meters measured water velocity, direction, temperature and conductivity with 10 min intervals during 38 days.

2. Water sampling was made in 8 sites which included the current measurement verticals. Sampling was done in different levels in order to get the concentration profiles. There were 10 sampling times, five of them were during four consecutive days. This made it possible to see the rapid changes in water quality. From the samples temperature, conductivity, total P, phosphate P, nitrogen, chlorophyll-a, secci-depth, pH, turbidity and oxygen were analyzed.

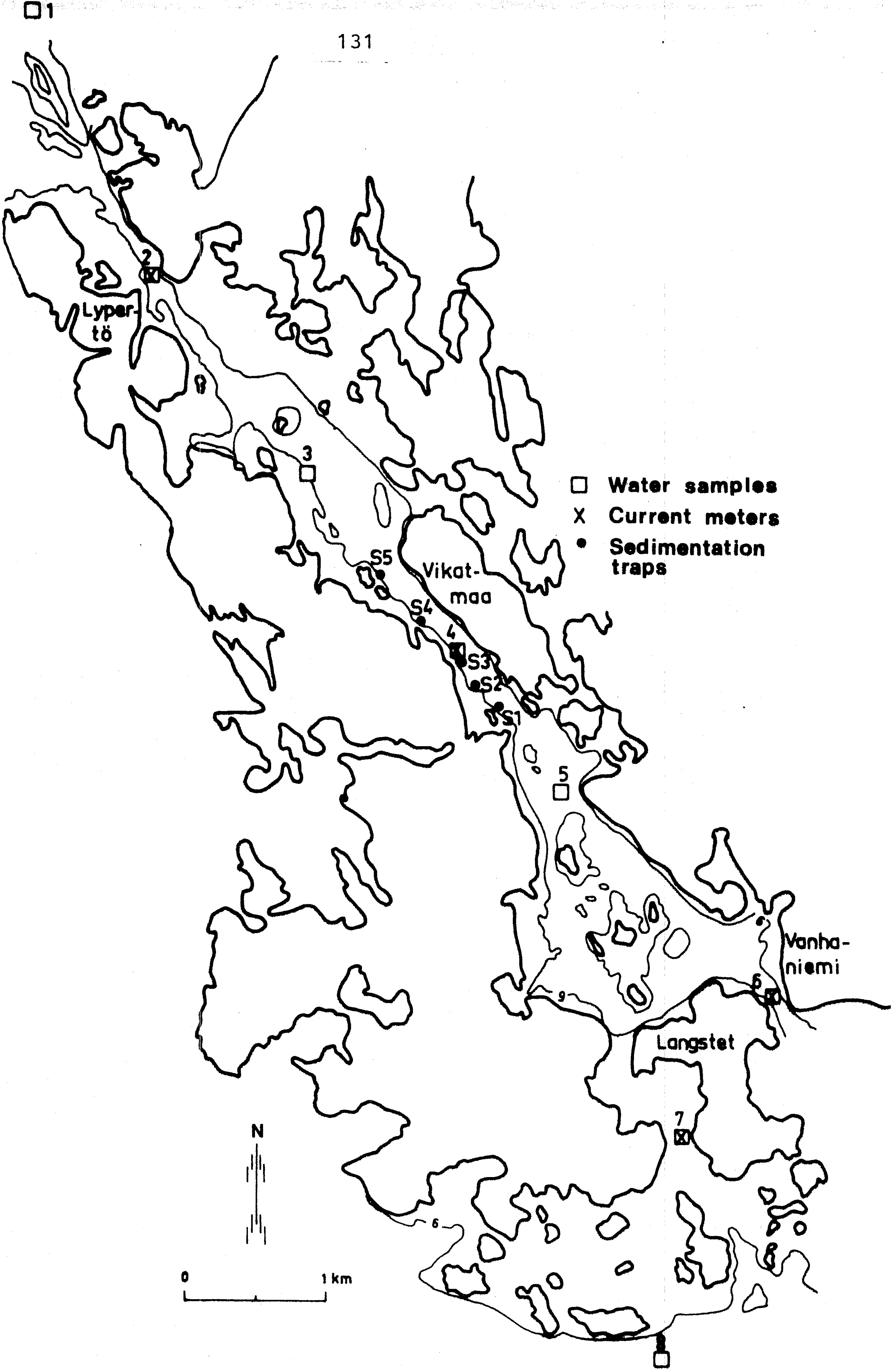


Fig. 1. The Ströömi Sound and the measurement points.

Results

The maximum observed velocity was 60 cm/s. The flow directions varied with different periods (Fig. 2). The average flow in the lower layer was towards SE and in the upper layer towards NW. The boundary between the layers was in six meters depth. The daily means of the velocity components depend on the northern component of the wind and also on water level variations. The correlation coefficients squared varied between 0.170 and 0.674. With the long period wind distribution and the obtained equations in the two SE-sounds it was possible to calculate the average water excange during the summer with normal winds and water levels.

Sedimentation traps showed greatest values towards NW from the fish farm (Fig. 3) . There was a one day period (period 5) with strong currents. During that day maximum values were obtained in all traps. This was a clear evidence of resuspension. The main sedimentation area was between the points 4 and 5.

The relationship between phosphorus concentration and current velocity in Ströömi made it possible to give a rough estimate of the phosphorus balance in Ströömi. During the 38 day period 14 tons phosphorus left Ströömi. Sea water brought 12 tons, from which 9 tons was in the upper 6 meter layer. The phosphorus amount in the water mass in Ströömi grew with 1.7 tons. So Ströömi produced 3.7 tons phosphorus during the period. This is fairly close to the approximated load from fish farm (5.3 tons).

CASE 2. PIETARSAARI

Pietarsaari is a town on the NW-coast of Finland with 30000 inhabitants. There are the pulp and paper factories of Willhelm Schaumann ltd. The factories are situated on the shore where water is relatively shallow (Fig. 4). The enviromental effects of the factories during the years have been rather obvious. During the recent years the firm has invested much in a new waste water treatment plant. New studies about the waste water spreading was needed in order to determine the effects of the loads coming from the factories, from the municipal treatment plant and also from the Lake Luodonjärvi.
The main questions were:

-what determines the mixing in the area? (wind, flow from Lake Luodonjärvi, water level changes, the mean Bothnian Bay current)
-where does the waste waters spread in calm, in windy and in winter conditions?
-how much and how quickly the loads from the factories affect the water quality of the area?

The two dimensional water quality model VENLA (Eloranta & al. 1981) was selected to simulate the water quality on the area in different water and meteorological conditions. For the model calibration

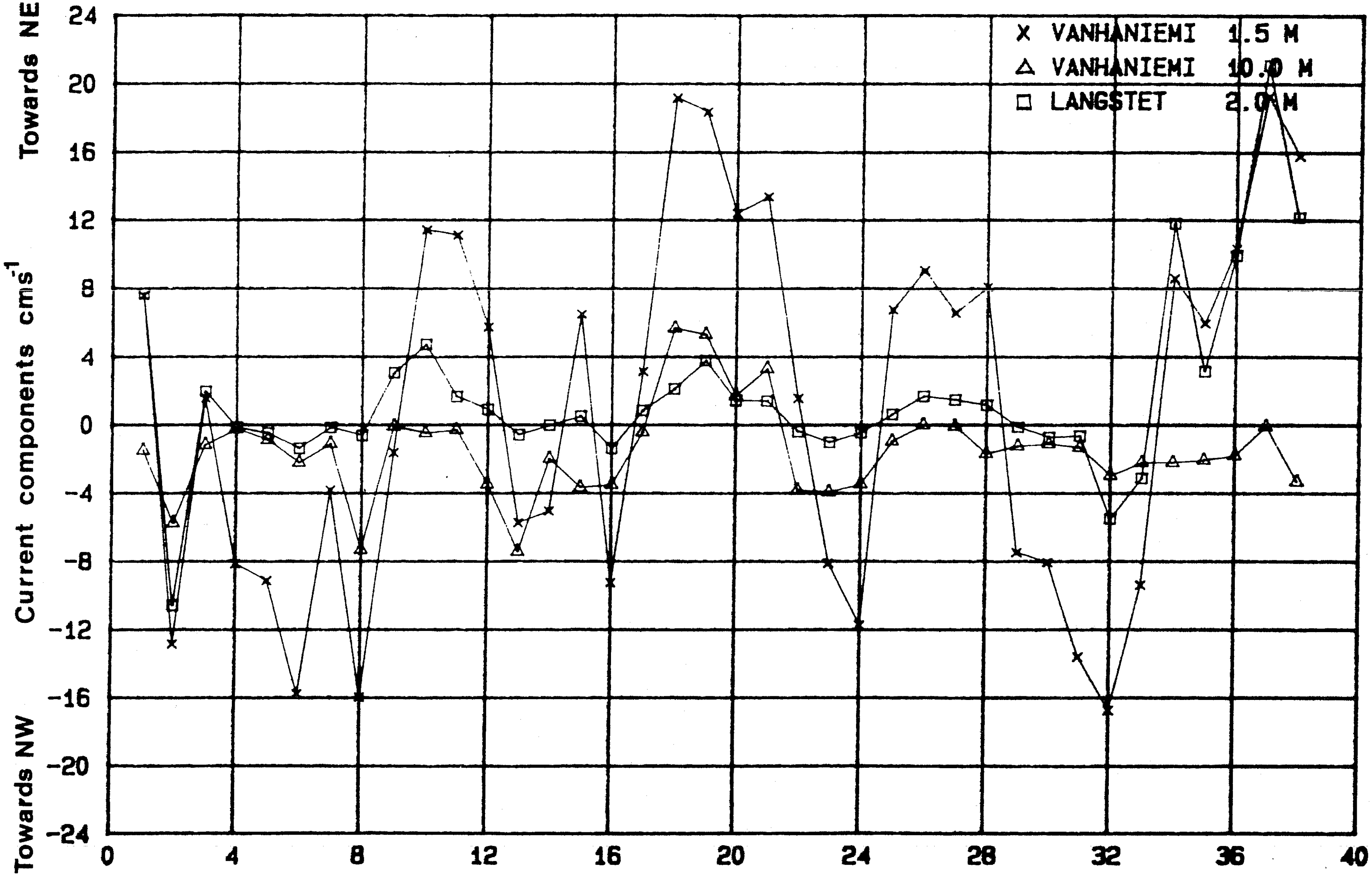


Fig. 2. The daily means of water currents in points 6 and 7 in Ströömi.

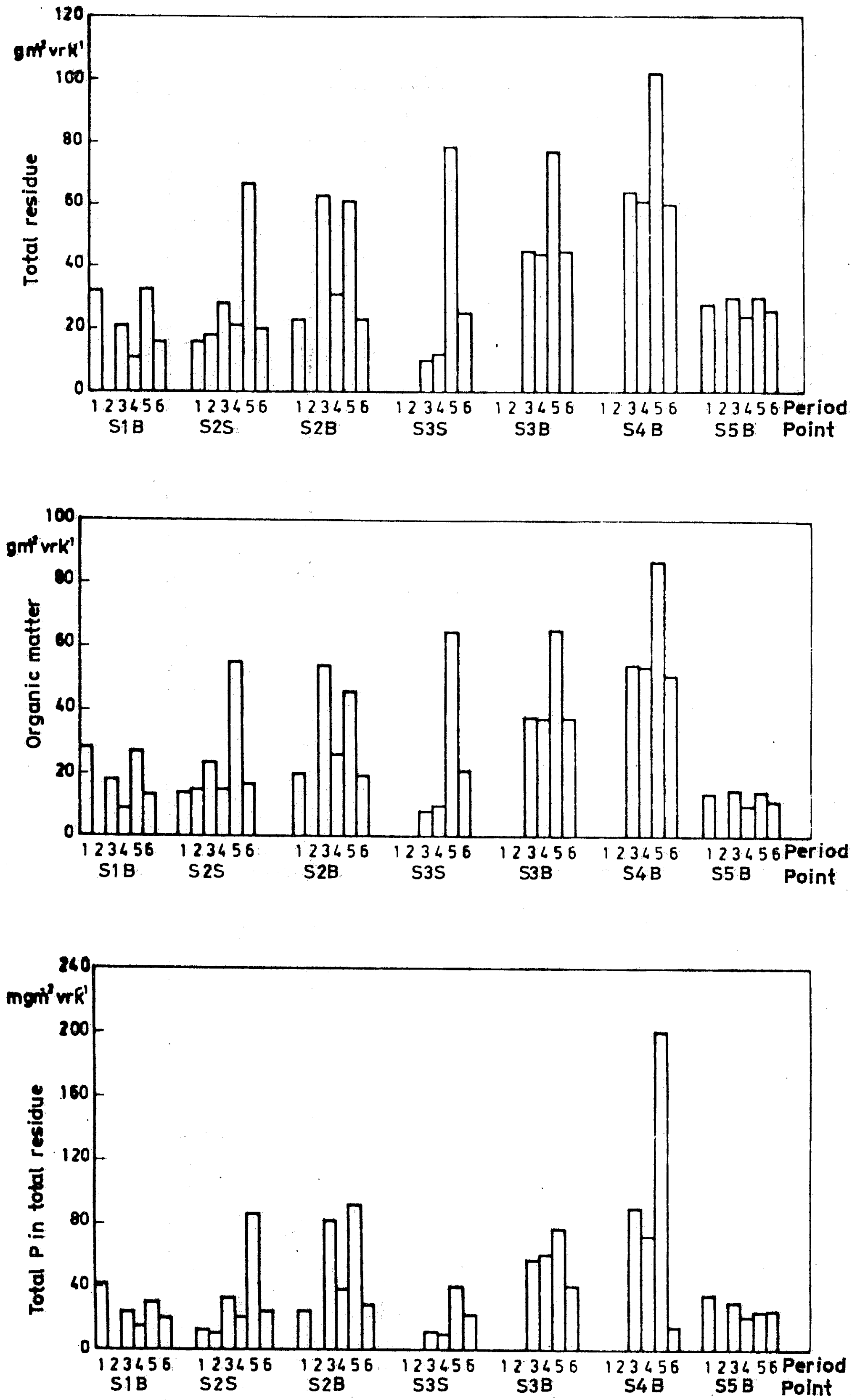


Fig. 3. The amounts of trapped material during the measurement periods. S = surface trapps, B = bottom trapps

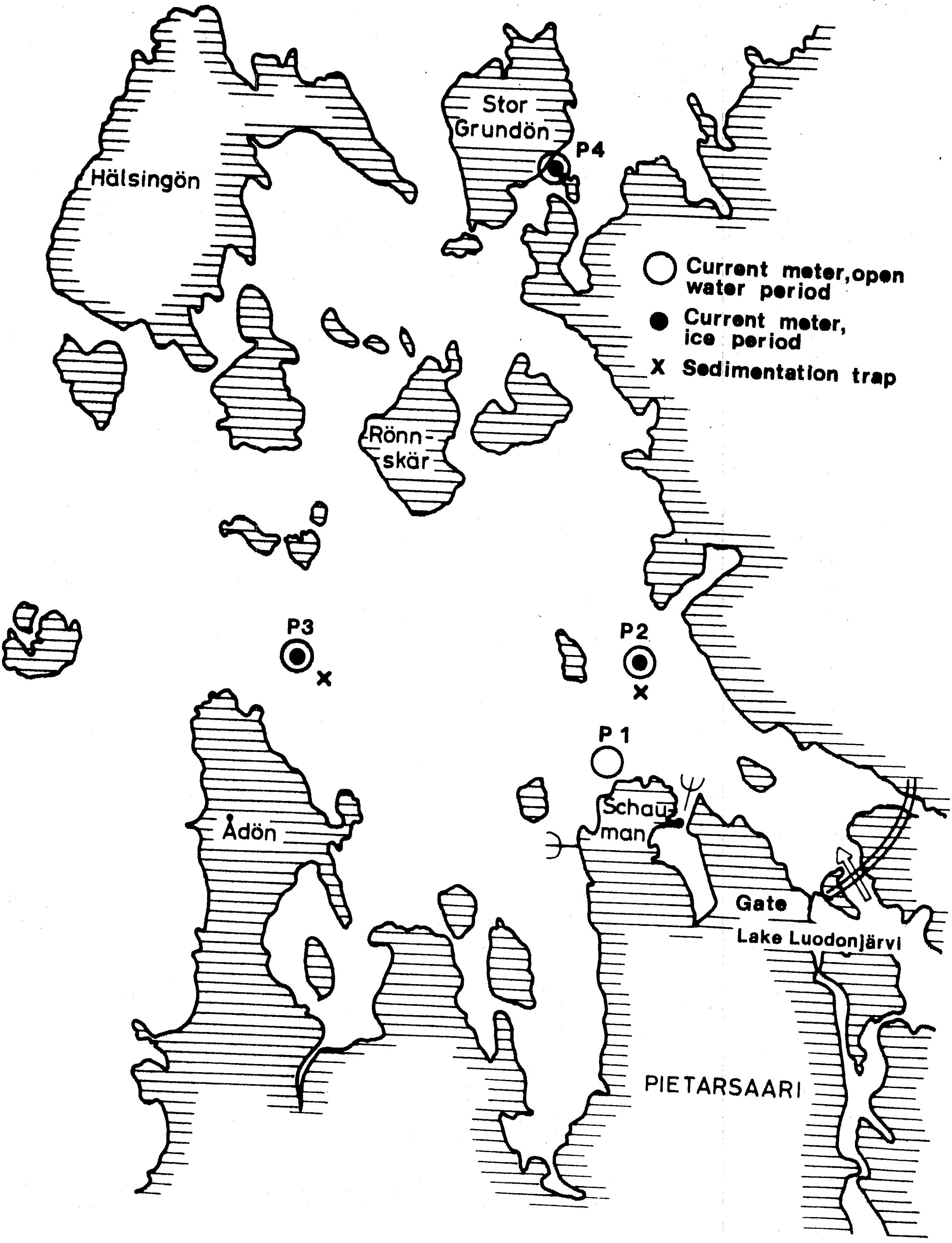


Fig. 4. The investigation area in Pietarsaari.

and verification the following measuring program was established:

1. Current mesurements in four measuring sites. During the open water period in Oct 1985 there was two measuring sites on the outlet area of the factories, one in the deep opening towards the sea in the west and one meter in the shallow sound leading to the north. Trials were made also to measure currents in the gates leading from the Lake Luodonjärvi, but this did not succeed. During the ice covered period in February-March 1986 the same sites were used, except for one close to the warm outlet with bad ice conditions.

2. Wind obeservations were made on the Mässkär-Island eigth times a day. Hourly water level observations were made in the habour.

3. The discharges from the Lake Luodonjärvi were calculated on the basis of the opening time of the gates and the mean observed velocity in the gates. A current meter was also installed in the gate opening but it was damaged in the heavy current with ice.

4. Sedimentation traps were placed in two sites during both periods. There purpose was to get sedimention values for the bottom interaction in the water quality model. The traps were placed 0.7 m above the bottom. There were three trapping periods during the open water period and two during the ice covered period. The length of the periods varied from 7 to 27 days. From the trap samples suspended solids, phosphorus in suspended solids, and the organic matter were analyzed.

5. Water samples were taken during the measuring periods from the existing water quality control sites and also from the current measuring sites.

6. The loads from the factories were available as daily mean values.

Results

The velocity-direction frequency distribution showed that the current direction during the open water period was in P1 and P3 near the surface mostly towards the sea.At the bottom in P3 35 per cent of the observations showed a current towards the shore. In P2 the velocities were smallest and the distribution was most even. In this point the current was towards the sea 58 per cent of the time and rest of the time towards the shore. The greatest velocities were also measured here. The currents were clearly depenent on the wind, seven of the ten daily velocity components were statistically extremly dependent on the wind components or the sea water level varations. The mean value for the correlation coeffiecient squared was 0.669. The regression equations gave a very good basis for the forecasting of the velocity vectors during different winds and sea levels. By this way it was possible to calibrate the numerical flow field calculation in the VENLA-model for the open period.

During the ice covered period the velocities were small. The greatest values were again obtained in P4 where the mean velocity was 8.2 cm/s. In other points it was between 2.2 and 3.2 cm/s. The regression between velocity components and water level changes and flow from Lake Luodonjärvi was much weaker than the regressions during the open water period. Only three from the eigth components were explained by these variables.

The VENLA is a two dimensional depth integrated model. The grid size in this application was 500 m.The whole mesh consists of 20 x 30 grid points. The flow field model is calibrated mainly with the eddy viscosity coeffiecient. When the flow fields are simulated well, the work with the transport and water quality model starts. Because the general water quality of the area is of interest and the area is shallow the depth integrated model is a suitable tool to predict the water quality. The selection of the calibration points and the calibration layer depth was based on the following observations about the transport of the effluents:

-each point represents a certain sub area
-during the open period there is no vertical stratification and the mean value of the concentration in each vertical is used
-during the ice covered period the surface concentration is used, because the effluents eem to be transported in the lake water entering the area
-during the ice covered period the flow from the Lake Luodonjärvi strongly affects to the spreading of effluents

This project is going on and the results will be reported at the end of this year.

CASE 3. LAKE PYHÄJÄRVI IN SÄKYLÄ

This lake is the most remarkable lake in the South Western Finland. Its area is 154 km^2 and the volume about 0.85 km^3. This means that the average depth is 5.5 m. It is exceptionally open and has only a few islands. The water quality is very good and the lake is also very productive for Finnish conditions. The yearly fish yield is 63 kg/hectar, which is about seven times the mean fish yield of a Finnish lake. The town Turku, situated about 60 km from the lake, has shown interest to take water from the lake. The discussion about this plan and about its affects on the lakes biology has been going on during the last 20 years. The local population is very actively against the plan. As a consequence of this the lake has been under an intensive research. Kuusisto (1975) has studied the water balance of the lake.

The question raised concerning lake hydrodynamics were:

-What kind of circulation is there in the lake?
-What determines the water excange between the different parts of the lake?
-What is the affect of the intake (1 m^3/s) to the circulation?

The currents have been measured during the ice covered period in winters 1977 and 1979 (Sarkkula and Forsius 1977, 1979). During the open period currents heve been measured in the autumns 1980 and 1986.

The currents were hardly detectable in the winter. The open water measurements were made in 1980 in three points (Fig. 5) with altogether 4 meters. The results revealed that there are conside-

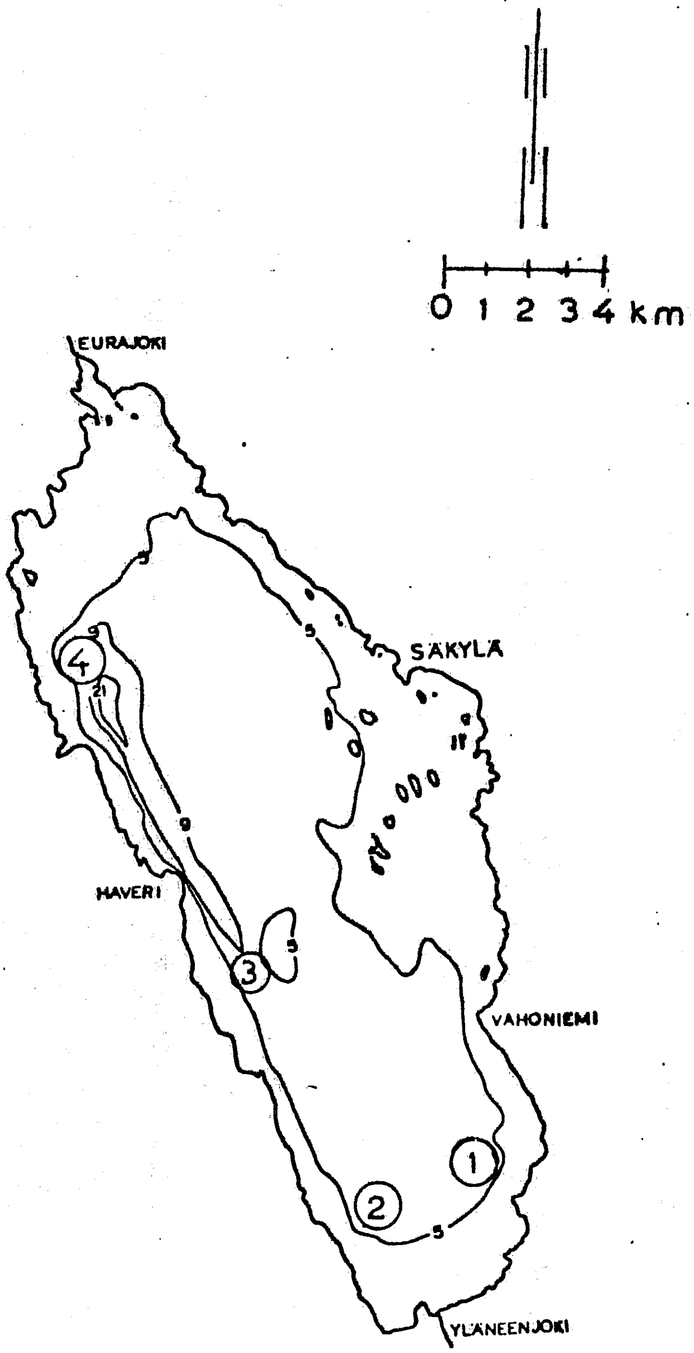


Fig. 5. Lake Pyhäjärvi in Säkylä and the water current measurement points in the autumn 1980.

rable currents, the maximum velocities varying from 25.0 to 35.4 cm/s. The mean wind velocity measured in Turku airport was 3.4 m/s. The wind distribution was geared towards the east from the normal October distribution. The depency of the current components on wind was anyway rather weak. The correlation coefficient squared in the regression equations varied between 0.25 and 0.36. Only with some certain wind directions the current velocities can be forecasted from the wind.All this showed that the flow patterns in the lake were much more complicated than one could expect on the basis of the lake morphometry. The flow fields on different wind directions were calculated with a two dimensional depth integrated model. The grid size was 1000m. The wind velocity used was 5 m/s. The simulations showed that there are several gyres with differnt scales and varying directions. On the basis of energy studies it was pointed out that during the open water period the wind determines the whole circulation and the effect of the water intake is minimal. During the ice covered period the effect is seen but limited to the vicinity of the intake pipe.

The studies of the lake circulation are continued during the year 1986 in the framework of the cooperation between NBWE and VITUKI Hungary. The essential question for the understanding of the lake response to the wind is the current observations in the representative points. For this new calculations of the flow patterns with two dimensional ADI-model was done. The grid size was now reduced to 500m. The size and location of the different gyres was now determined more correctly (Fig. 6). On the basis of these and also the previous results the location of the eigth current meters was decided. Also the wind measurements were done on the lake with automatic instruments. The measuring period was from the beginning of Sept. to the middle of Oct. 1986. This data is studied by statistical approache including the power spectrum in order to find out the current causing factors and the oscillations in the current field. The investigation is completed in the February 1987.

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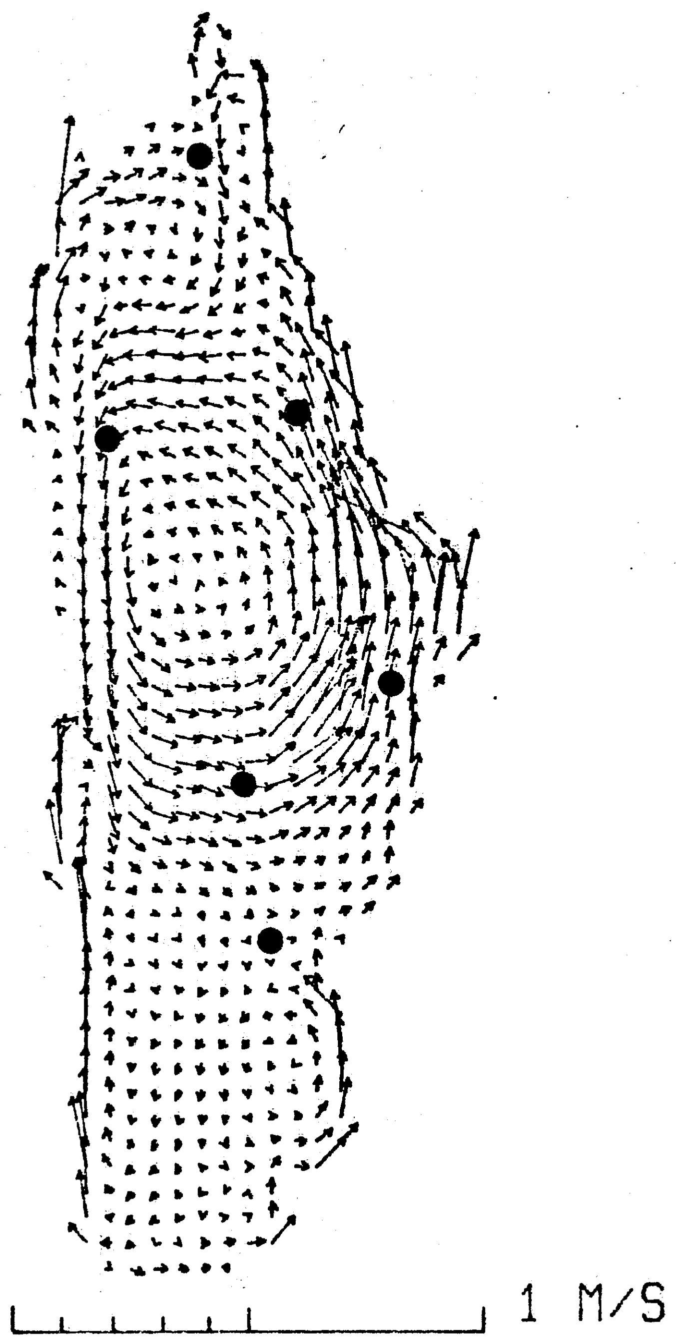

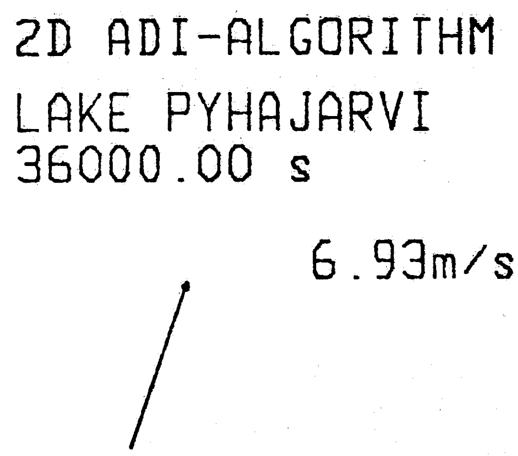

Fig. 6. A simulated current pattern in Lake Pyhäjärvi and the measurement points in the autumn 1986.

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DATA REQUIREMENTS OF WATER QUALITY MODELS

Jorma Niemi

ABSTRACT

Water quality models need data for the definition of the initial conditions of the ecosystem, for calculating the material loads to the water body through tributaries, precipitation and waste-water treatment plants and for meteorological phenomena. Data are needed also for the calibration and validation of the model. The data typically include hydrological data, water quality data and meteorological data, which must be gathered either by using observation networks monitored regularly or in addition to this by carring out special investigations. The gathered data have to be processed in a suitable form for the model by using special computer programmes made for this purpose. The data needed by the simulation models is often divided into initialization data and simulation data. The general requirements of data in models, sources of data and processing of data are presented and discussed on the basis of the experiences obtained in developing and applying a simulation model for Finnish lakes.

1. INTRODUCTION

Environmental systems are frequently investigated by using mathematical models that are based on the information available of these systems. Water quality models are one group of these models and they are typically used for obtaining understanding of the basic phenomena of the ecosystem, for making predictions of the water quality e.g. with alternative loadings or with varying meteorological data and as an aid in planning. The structure and mechanisms of the models vary according to the objectives for which they are made. Their common feature however is that they need data of the system that is investigated for defining the initial conditions of the ecosystem and for the calibration and validation of the model. The data must be collected by using either permanent networks used for observations or by carring out additional investigations according to the requirements of the model. The type of data needed varies according to the nature of the model. In an empirical model calculating e.g. the average phosphorus concentration of the lake the data needed includes information of the morphology of the lake, input loading of phosphores to the lake and observed concentrations in the lake. Examples of empirical water quality models are e.g. Dillon and Rigler (1974), Vollenweider (1975), Frisk et al. 1980 and OECD (1982). In complicated simulation models (Park et al. 1975, Niemi 1979, Kinnunen et al. 1982) however the data requirements are larger and include e.g. data of meteorological phenomena, discharges and concentrations of all simulated state variables of all the incoming waters such as tributaries, wastewater treatment plants, diffuse loading and precipitation and historical data of the state variables in the water body. Some models may even require data of the soil of the discharge area if diffuse loads are calculated by the model and not given as driving variables or if the models include other factors than water quality (Knisel 1980).

Besides this fundamental division there are several other ways of grouping water quality models on the basis of their characteristics (Table 1). Depending on the structure of the model it can belong simultaneously into several groups e.g. be one-dimensional, deterministic model to be used in water management. This classification can be helpful in evaluating the model structure but less appropriate from the point of view of data requirements. However this division can give insight to the data requirements as well e.g.- descriptve models typically require more than data black box models, and a three dimensional model more than one-dimensional if they are otherwise similar.

The objective of this paper is to investigate briefly the data requirements of various types of water quality models and to evaluate how this data can be obtained in applying the models. The main empasis is laid on the water quality data. The models need besides water quality data also data of the values of parameters. The question of parameter estimation however is not treated here.

2. DATA REQUIREMENTS

2.1 General

The data requirements of the models vary depending or the type of model. Typical data include generally hydrological data, water quality data and meteorological data (Table 2). These models represent relatively complicated models with many forcing functions or driving variables and state variables and large data requirements. Empirical models however can generally be applied with much less data.

In simulation models, such as the ones presented in Table 2, the data is divided into initialization data and simulation data. Initialization data is composed of data used to define the initial conditions of the lake when simulation is started and to specify computer program options. This data includes morphological data, values for parameters, initial values for the state variables and program controls such as the lenght of simulation step, number of tributaries etc. Initialization data is given to the model only once at the beginning of the simulation. Simulation data includes the data of the quality and quantity of all the inflowing waters such as tributaries, wastewater treatment plants, precipitation and meteorological data. Simulation data is given to the model once during every simulation step, e.g. once in every eight hours.

2.2 Hydrology

Hydrological data is needed in some extent practically in every water quality model. In empirical models (Frisk et al. 1980) the data requirements are relatively small as usually only discharges of the tributaries, wastewater treatment plants and possibly the runoff values are needed. As the structure of the models becomes more complicated, especially the structure of the hydrological module, the amount of data increases. In one-dimensional models (Niemi 1977, Kinnunen et al. 1982) the hydrological part is treated in a relatively simple manner and the hydrological data requirements are not substantially larger than those of empirical models. However in two and three dimensional models the hydrological information becomes more important as the hydrological resolution becomes higher (Virtanen et al. 1986). These models generally need data of discharges of incoming and outcoming waters, water levels and water currents.

The data readily available from data banks can in some cases be sufficient for empirical and one-dimensional models. In empirical models the data necessary for the calculation of material loads is most difficult to obtain. For two and three dimensional models however special field measurements for estimating the water currents must be carried out. This has to be taken into account when the application of the model is planned as the gathering of the data takes at least one year, but in practice several years before sufficient material for the calibration and validation is available.

2.3 Water quality

The number of state variables in the water quality models may be quite large, from one or two state variables to of the empirical models up to about twenty or even to about thirty variables (Table 2). Observed data of all the state variables is needed for the definition of the initial conditions as well as for calibration and validation. The frequency of the observation carried out in the real water body varies. Optimal frequency would be a measurement during every simulation step, which is in practice impossible. The initial conditions prevailing in the lake during the spring turn over are normally given to the model as the water mass during this time is homogenous and the initial conditions can be obtained with relatively few measurements. The model output is compared with the observations of those days of which best set of measurements is available. The representativeness of the observations defines to a great extent how reliable the comparisons between the model output and observations will be. Expecially during the calibration the observed data set should be sufficient so that calibration could be made with good data set. The same holds true of validation as well, but without proper calibration the validation process may not be reliable. Typically the basic water quality data needed for large models is available only of the most important Finnish lakes. The basic data set must in practice be supplemented with additional measurements, normally with biological observations. If possible the data set used for calibration must be completed too. As an example of the data sets used in the National Board of Waters and Environment are the applications of a model to Lake Päijänne (Kinnunen et al. 1982) and to Lake Pyhäjärvi (Niemi and Eloranta 1984). In the former four sets of data, each one covering one year were processed and for the latter two sets of data.

2.4 Meteorology

Meteorological data is normally needed in simulation models where it is used as driving variables. This data includes temperature (wet and dry bulk) precipitation, air pressure, wind speed and cloud cover. The data are normally obtained from the meteorological observations carried out in a suitable place, preferably near to the case study water body. As the data are used as forcing functions a data set must be defined for every simulation time step. Normally the observed data do not coincide to the same intervals and processing of data are necessary. Accuracy of this data set is crucial as the driving variables form the power that drive the model towards the following time step.

3. SOURCES OF DATA

The nationwide collection of the data of the Finnish water bodies is carried out mainly by the National Board of Waters and Environment by using the networks covering the whole country. The networks include e.g. a group of Finnish rivers and lake deeps. The gathered information is composed of the data of water quality. hydrology and the data of biological water quality variables e.g. the species composition and biomass of phytoplankton. The information is transferred to the data banks of water quality and toxic substances, to the bioregister including data of phytoplankton, specimen banks for water and fish samples, data banks of water level and discharges. The data banks are presently being developed very fast and new ones such as a lake register is under construction. In the near future all these data banks will become a part of a large environmental system that is being developed. The new system will include all the environmental information of the country.

The information stored in the data banks expanses rapidly. This data offers good apportunities for applying various methods including water quality modeling so that the information could be used more effectively by planners and decision-makers. The use of models as a method for processing this information is strongly couraged.

The data banks and especially the new environmental information system under construction are a firm background to the data needs of the water quality models. The data available in the data banks may not, however be sufficiently frequently sampled and may not contain all the necessary information. Typically the data available from the data banks is supplemented with additional data gathered by special field studies for the application.

4. PROCESSING THE DATA

The data gathered to be used in models is seldom in a form required by the model. The data has therefore to be processed to be suitable for the model.

Hydrological data that are typically used as driving variables must be specified for each simulation time step, which is normally shorter than the interval of observation carried out in the field. The data must be interpolated on the basis of obsevations or it must be kept constant during certain time intervals. Eg. the discharges of the tributaries can be kept constant during a week or even a month depending on the time of the season. Similar approach is necessary in handling diffuse loading and precipitation data. Generally the discharges of the wastewater treatment plants are recorded frequently and in processing this data e.g. the discharge values may not need interpolation at all.

Water quality data are used in the models as initial values of the state variables describing the state of the ecosystem and also in calculating the material loads to the water body via precipitation, diffuse load and through wastewater treatment plants. In the calibration and validation the observed values of the water quality variables are used such as they are. The model is made to calculate the values of the state variables for those days of which observed material is available.

In processing the meteorological data for the model interpolation and extrapolation of the observations is necessary as the observations do not coincide to the same time intervals used in the model. Meteorological data are used as driving variables and therefore the amount of this data may be considerable in long simulations. If the model is used for predictions the data set of driving variables must be prepared. Real data of some years can be used or typical data sets of e.g. dry, wet, warm and cold years can be used. Regardless of the type of data is used its preparation takes time.

Small computer programmes are of help in the preparation of data sets of models. Especially if the same model is used the time spent in constructing special programmes for the handling of data is well used and comes back as data are prepared for the running the model.

5. DISCUSSION

Water quality models are abstractions of real ecosystems. Their output must be compared with the observations made of the real ecosystems in calibration and validation. For this process observed data are necessary.

In the nature there are several types of variation, e.g. in the lake there are variations in water quality both in time and space. There is long term variation, short term variation and spatial variation. Errors due to the transportation and analysing of samples are in the final values of water analysis. These phenomena may distort the picture of water quality of the lake and they cannot be taken into account by analysing a single water sample. If samples are taken frequently this variation decreases. Often, although this variation is known, observations are treated as correct, absolute values, that are a basis for tuning the model. These facts should be considered in comparing the model output with observations.

The effect of data in models is extremely important. However it is often assumed that the information content of the model increases with increasing data only to a certain level after which even if the data are increased the information content stays on the same level. This may be due to several factors. One explanation may be that there is a certain limit until which the accuracy of the model is data limited, and after that the basic mechanisms of the model such as the correcteness of the equations, values of parameters etc. should be further developed to improve the model output.

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Table 1. An example of possible divisions of water quality models

Research	- management
Distributed	- lumped
Linear	- non linear
Stochastic	- deterministic
Dynamic	- steady state
Descriptive	- black box
One dimensional	- three dimensional

Table 2. Comparison of the data requirements of some water quality models (Niemi 1977, National Board of Waters 1978).

		Model					
		Delaware Estuary	Narragansett Bay	Lake Washington	Phytoplankton model	Lake George	Lake Esrøm
Reference[1)]		1	2	3	4	5	6
Forcing functions	Air temperature (dry bulb)		x	x	x		
	Air temperature (wet bulb)			x			
	Water temperature	x	x		x	x	x
	Solar radiation	x	x	x	x	x	x
	Turbidity	x					
	Toxic substances	x		x			
	Hydrography	x	x	x	x	x	x
	Aeration constant[2)]	x	x	x	x	x	x
	Nutrients	x	x	x	x	x	x
	Air pressure			x			
	Cloud cover			x			
	Wind speed			x			
	Data on tributaries				x		
	Photoperiod				x		
	Extinction coefficient				x		
	Water travel/streamflow	x	x	x	x	x	x
Forcing functions		8	7	12	9	7	6
State variables	1. Biotic						
	Bacteria	x		x			
	Phytoplankton	x	x	x	x	x	x
	Zooplankton	x	x	x	x	x	x
	Benthic animals		x	x			
	Macrophytes					x	
	Fish	x		x		x	x
	2. Abiotic						
	Water temperature			x			
	Oxygen	x		x			x
	BOD	x		x		x	
	Alkalinity			x			
	pH			x			
	CO_2			x			
	Total dissolved solids			x			
	Detritus		x	x			
	Sediment			x			
	Nitrogen (undefined)					x	
	Nitrogen (Kjeldahl)	x					
	Organic nitrogen				x		
	Detritus nitrogen					x	x
	Inorganic nitrogen					x	x
	NH_3		x	x	x		
	NO_3		x	x	x		
	NO_2		x	x			
	Phosphorus (undefined)						
	Phosphate phosphorus		x		x	x	
	Detritus phosphorus						x
	Organic phosphorus			x			
	Inorganic phosphorus				x		x
	Total phosphorus	x					
State variables		8	8	18	7	10	8
Forcing functions + State variables		16	15	30	16	17	14

1) 1. Kelly (1974)
   2. Nixon and Kremer (1977)
   3. Chen and Orlob (1972)
   4. O'connor et al. (1975), Ditoro et al. (1975)
   5. Park et al. (1975)
   6. Gargas (1976)

2) Often calculated from other data

PLANNING MODEL OF WATER RESOURCE STRUCTURE AND PARAMETORS TAKING ACCOUNT OF WATER PROTECTION MEASURES

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Khranovich I.L.

Introduction

A water resource system (WRS) involves an assembly of interacting sources of water, reservoirs, seas and water users, connected by river and canal reaches.

A WRS coordinates water demands with available water resources. According to this function of the system, its parameters need to be coordinated in the best way. In the particular work under the term "best" or "optimal" such WRS parameters and WRS operational regimes are meant, with which national economic expenses, for the design period, reduced to a comparable form, are minimized. Costs involve operating expenses and capital investments. Effect of water use is also taken into account in them.

When considering the selection of WRS parameters, it is supposed that peremeters of elements of the existing WRS (volumes of reservoirs, canal capacities and efficiency, production capacities, i.e. capacity of interprises-water users and purification facilities, etc.) and probable regimes of water and impurities entry into a system are known. Reconstruction elements and elements under construction with possible values of their parameters, received in the result of a preliminary, are thought to be known too. Solution of this problem consists in selecting an optimum set of parameters of WRS elements among probable ones.

It requires a great number of possible sets of elements' parameters and relevant operating regimes of the system to be compared. To solve the problem an optimization model is used, where all permissible sets of versions are compared. Water resources distribution and control of their quality are analysed jointly in the proposed model. Such an approach unlike the traditional separate analisis of quantitative and qualitative characteristics of water resources allows all water management measures to be solve in one complex.

The system operation is assumed in discrete time. The model takes account of random process of water and impurities entry and utilization which is presented by a finite set Ω of characteristic realizations of conditions ω .

A WRS is presented by a network Γ (J,S), the configuration of which corresponds to a diagrammatic representation of WRS. In the network Γ (J,S) all the assumed WRS elements, existing or possible, are represented. Elements of Γ (J,S) have characteristics of their own, their interaction is produced by a movement of a flow, corresponding to water and admixture flows in the modelling system.

For any WRS element there is singled out a finite set of possible vertions of its development, each of them is characterized by a set of element's parameters.

In the model each vertion of WRS element's development is presented as an operational element. Operation of the z -th element according to z_α -th vertion of the development is described by a characteristic function of the vertion $b_{z\alpha}$ =1, if the z_α -th vertion of development is accepted; $b_{z\alpha}$ =0, if the z_α -th vertion is not accepted. As a WRS element can operate according to one vertion of finite sets a_z , then characteristic functions of element's development variants are linked by the relationships

$$z_{z\alpha} \in \{0;1\}, \sum_{z_\alpha \in a_z} z_{z\alpha} = 1 \qquad (1)$$

<u>River and canal reaches and water users</u>, except those using water from reservoirs without its divertion and considered as a single whole together with reservoirs, are depicted by Γ (J,S) arcs with gain and retardation.

The flow $q^{\omega}_{s\alpha}$ at the beginning of the S_α -th arc corresponding to the S_α -th varient out of the multitude a_s of possible development variants of the S -th users under ω stochastic conditions, is linked with the flow $q^{K\omega}_{s\alpha}$ at the end of this arc by the equality:

$$q^{K\omega}_{s\alpha t} = K^{\omega}_{s\alpha t} \; q^{\omega}_{s\alpha, t-\theta} \qquad (2)$$

without non-negative retardation $\theta^{\omega}_{s\alpha t}$ and an amplification factor, having the meaning of a ratio of water, returned by the user, to the amount of water, allocated to the user. Its values are within the range of $1 \geqslant K^{\omega}_{s\alpha t} \geqslant 0$.

In the model users' requirements to water resources quantity and water consumption restrictions cause appearance of sets of possible values of arc flows:

$$\underline{q}^{\omega}_{s\alpha t} \leq q^{\omega}_{s\alpha t} \leq \bar{q}^{\omega}_{s\alpha t} \qquad (3)$$

<u>~~Water-sources~~ with uncontrolled consumption</u> (for example, a natural river run-off) in the model are presented by the flow sources of prescribed intensity b^{ω}_{it} located at $i \in J$.

<u>Water sources with controlled consumption</u> are described by

fragments of Γ (J,S), each having vertices, the number of which equals the number of vertions of source's development. Such sources include, for example, transferance systems from other regions. Each vertex is connected with the remaining part of the network by outgoing arcs $S_{\alpha 1}$ and $S_{\alpha 2}$ with the gain coefficients $K^{w}_{s\alpha 1}$ =1 and $K^{w}_{s\alpha 2}$ =0.

Reservoirs and seas in the model are regarded as a single whole with users, located on them. They are expressed by storages, located in the graph's verticis. Their supplies $\theta^{w}_{i\alpha t}$ equal the amount of water in reservoirs and seas. These contraints cause sets of possible values of storage supplies

$$\underline{\theta}^{w}_{i\alpha t} \leq \theta^{w}_{i\alpha t} \leq \bar{\theta}^{w}_{i\alpha t} \tag{4}$$

In the model relationships between intensity of water losses in reservoirs and seas due to filtration and evaporation are approximated by linear ones.

$$\delta \theta^{w}_{i\alpha t} = \gamma^{w}_{i\alpha t} \theta^{w}_{i\alpha t} \tag{5}$$

Impurities and Water Resources Quality

In addition to "basic" flows $q^{w}_{s\alpha t}$ and supplies $\theta^{w}_{i\alpha t}$ expressing water storage and discharge, "additional flows" $y^{w}_{s\alpha \ell t}$ and supplies $Y^{w}_{i\alpha \ell t}$ simulating impurities (is the type of impurity), are included in the model. Interaction of these flows corresponds to representation of processes of impurities transition and transformation by a system of linear differential equatuons of the Streater-Pholps type [9].

In the elements of the network in question flows and supplies of impurities of each set should meet the following requirements

$$\sum_{\ell \in L_\nu} d^{\omega}_{z_\alpha \ell t} Z^{\omega}_{z_\alpha \ell t} \leq \beta_{z_\alpha t} x^{\omega}_{z_\alpha t}, \quad z_\alpha \in R = S \cup J, \tag{6}$$

The set of types L of impurities are divided into subsets L_ν which can have an unempty crossing, according to a limiting factor in such a way, that $\cup L_\nu = L$, where $d^{\omega}_{z_\alpha \ell t}$ is magnitude, inverse to the ultimate permissible concentration $\bar{C}^{\omega}_{z_\alpha \ell t}$ of ℓ-th admixture in the z_α -th element of Γ (J,S) under stochastic conditions ω if $z_\alpha \in S$ $Z^{\omega}_{z_\alpha \ell t}$ denotes an impurity flow of ℓ type in the z_α arc $y^{\omega}_{z_\alpha \ell t}$, where $z_\alpha \in J$ $Z^{\omega}_{z_\alpha \ell t}$ - impurity supply of ℓ type in the z_α -th storage $Y^{\omega}_{z_\alpha \ell t}$; $x^{\omega}_{z_\alpha t}$ denotes a water flow $q^{\omega}_{z_\alpha t}$ and supply $Q^{\omega}_{z_\alpha t}$. If $z_\alpha \in S$ the conditions (6) correspond to arcs' inlets and reflect users' requirements to quality of water, allocated to them. Requirements to quality of water, returned into the system, are prescribed by similar conditions, imposed on the impurity composition at arcs' outlet.

$$\sum_{\ell \in L_\nu} d^{\kappa\omega}_{s_\alpha \ell t} y^{\kappa\omega}_{s_\alpha \ell t} \leq \beta^{\omega}_{s_\alpha t} q^{\kappa\omega}_{s_\alpha t}, \quad s_\alpha \in S \tag{7}$$

Moreover, the values of impurities flows and supplies have lower limits:

$$Z^{\omega}_{z_\alpha \ell t} > \underline{Z}^{\omega}_{z_\alpha \ell t}, \quad y^{\kappa\omega}_{s_\alpha \ell t} \geq \underline{y}^{\kappa\omega}_{s_\alpha \ell t}, \quad z_\alpha \in R, \quad s_\alpha \in S \tag{8}$$

The constants $d^{\omega}_{z_\alpha \ell t}$, $d^{\kappa\omega}_{s_\alpha \ell t}$, $\underline{Z}^{\omega}_{z_\alpha \ell t}$, $\underline{y}^{\kappa\omega}_{s_\alpha \ell t}$ in their physical meaning are non-negative.

It is worth mentioning, that conditions (6) and (7) impose tougher requirements on the impurities compositions, than non-excess of impurities concentration $C^{\omega}_{z_\alpha \ell t} = Z^{\omega}_{z_\alpha \ell t}/x^{\omega}_{z_\alpha t}$ and $C^{\kappa\omega}_{s_\alpha \ell t} = y^{\kappa\omega}_{s_\alpha \ell t}/q^{\kappa\omega}_{s_\alpha t}$ of all the impurities taken separately

From the assumptions about complete practically momentary mixing and linearity of impurities composition change it follows that impurities supplies in storages satisfy the system of equations:

$$Y^{w}_{i\alpha\ell,t+1} = \sum_{\gamma\in L} A^{\gamma\ell w}_{i\alpha t}\, Y^{w}_{i\alpha\gamma t} + h_t\, y^{w}_{i\alpha\ell t} \tag{9}$$

where $A^{w}_{i\alpha t} = \{A^{\gamma\ell w}_{i\alpha t},\ \gamma,\ell\in L\}$ is the nonsingular square matrix of substance transformation in the storage.

Solving the system of linear differential Streater-Phelps equations a relationship is obtained between the vectors of impurities at inlets and outlets of arcs, forming the set and representing in Γ (J,S) users without purification facilities, river and canal reaches.

$$y^{kw}_{s\alpha\ell t} = \sum_{\gamma\in L} A^{\gamma\ell w}_{s\alpha t}\, y^{w}_{s\alpha,t-\theta} \tag{10}$$

A functional relationship of type (10) between impurity flows at the inlets and outlets of arcs, representing users and sources with purification facilities in the model and forming the set $S_{11} = S - S_{1}$, is absent, since due to operational regimes of purification facilities for the same magnitudes $y^{w}_{s\alpha\ell t}$ different magnitudes $y^{kw}_{s\alpha\ell t}$ can be received. Impurity flows $y^{w}_{s\alpha\ell t}$ and $y^{kw}_{s\alpha\ell t}$ are interconnected by expenses of operation of elements with purification facilities.

Inevitability of receiving water with impurities brings about a requirement of a coincidence of a heterogenous flow composition at the inlets of arcs starting from the same vertex.

$$y^{w}_{s\alpha\ell t}\, q^{w}_{\tilde{s}\alpha t} = y^{w}_{\tilde{s}\alpha\ell t}\, q^{w}_{s\alpha t}, \quad \tilde{S}_{\alpha}, S_{\alpha} \in S^{1}_{i} \tag{11}$$

Continuity Equations

In the WRS water and impurities distribution subjected to the mass conservation law, in accordance with water and impurity flows in Γ (J,S) meet the system of flow countinuity equations

$$\sum_{i_\alpha \in a_i} \left[Q^{w}_{i_\alpha, t+1} - Q^{w}_{i_\alpha t} \right] = h_t \sum_{i_\alpha \in a_i} q^{w}_{i_\alpha t} = \qquad (12)$$
$$= h_t \left[\sum_{s_\alpha \in S_i} q^{\kappa w}_{s_\alpha t} - \sum_{s_\alpha \in S_i} q^{w}_{s_\alpha t} - \sum_{i_\alpha \in a_i} \delta Q^{w}_{i_\alpha t} + b^{w}_{it} \right],$$

$$\sum_{i_\alpha \in a_i} y^{w}_{i_\alpha \ell t} = \sum_{s_\alpha \in S_i^+} y^{\kappa w}_{s_\alpha \ell t} - \sum_{s_\alpha \in S_i} y^{w}_{s_\alpha \ell t} + b^{w}_{i \ell t} \qquad (13)$$

where S_i^+ is set of arcs, coming into the 1 -th vertex,

S_i^- is the set of arcs, outgoing from the -th vertex,

$b^{w}_{i\ell t}$ is the impurity flow of 1 type, coming into the i -th vertex under stochastic conditions w .

Resource Constants

The proposed model involves conditions, reflecting const - raints of labour, finantial and other resources and tasks on production targets, produced by water users. These conditions include constraints on the resources, connected with the WRS development and operation.

Resource constraints on the development are represented by a system of inequalities

$$\sum_{z_\alpha \in R} m^{d}_{z_\alpha} z_{z_\alpha} \leq M^d, \quad d \in \mathcal{D}_I, \qquad (14)$$

where M^d is the total amount of the d-type resource, which can be allocated for the WRS development of the region (capital investments, building materials, capacity of building organiza - tion etc.); $\mathcal{D}_I$ is the set of resource types taken into account, used in the WRS elements' construction and reconstruction; $m^{d}_{z_\alpha}$ the amount of the d -type resource, needed for the introduction of an α -version of development of the z -element of thw WRS.

In a functioning WRS restricted stored and unstored resources are used. Stored resources involve mainly utilised raw materials being used and such materials, as reagents for water purification. Unstored resources are labour resources electric energy, capacity resources of irrigation facilitie etc.

Employment conditions of stored resources cover the whole design period, and are described by a system of inequalities

$$\sum_{t\in[T_0,T]}\Big[\sum_{z_\alpha\in R}\varphi^{d\omega}_{z\alpha t}(x^{\omega}_{z\alpha t},u^{d\omega}_{z\alpha t})+\sum_{s_\alpha\in S_{\underline{II}}}\varphi^{\kappa d\omega}_{s\alpha t}(x^{\kappa\omega}_{s\alpha t},u^{\kappa d\omega}_{s\alpha t})\Big]\leq \tag{15}$$
$$\leq M^{d\omega},\quad d\in\mathcal{D}_{\underline{II}},\quad \omega\in\Omega$$

Here $M^{d\omega}$ and d have the same meaning as in (14); $\mathcal{D}_{11}$ is the set of stored resources,

$\varphi^{d\omega}_{z\alpha t}$ and $\varphi^{\kappa d\omega}_{s\alpha t}$ relationships of the resource magnitude being utilized and water quantity and quality in the element (φ) and returned into the system (φ^{κ}); $u^{d\omega}_{z\alpha t}=\sum_{e\in L}\delta^{d\omega e}_{z\alpha t}\,y^{\omega}_{z\alpha e t}$ and $u^{\kappa d\omega}_{s\alpha t}=\sum_{e\in L}\delta^{\kappa d\omega e}_{s\alpha t}\,y^{\kappa\omega}_{s\alpha e t}$ impurity complexes in the element and returned into the WRS, $\delta^{d\omega e}_{z\alpha t}$ and $\delta^{\kappa d\omega e}_{s\alpha t}$, non-negative numbers, reflecting the significance of the e -impurity in a complex.

Functions $\varphi^{d\omega}_{z\alpha t}$, $\varphi^{\kappa d\omega}_{s\alpha t}$ are covex in each of the variables $x^{\omega}_{z\alpha t}$, $u^{d\omega}_{z\alpha t}$ and $x^{\kappa\omega}_{s\alpha t}$, $u^{\kappa d\omega}_{s\alpha t}$; and are non-convex in their totality[4].

Unstored resources are used only in those time spans, where they are singled out. Conditions of their utilization correspond to these time spans and have the following form

$$\sum_{z_\alpha\in R}\varphi^{d\omega}_{z\alpha t}(x^{\omega}_{z\alpha t},u^{d\omega}_{z\alpha t})+\sum_{s_\alpha\in S_{\underline{II}}}\varphi^{\kappa d\omega}_{s\alpha t}(x^{\kappa\omega}_{s\alpha t},u^{\kappa d\omega}_{s\alpha t})\leq M^{d\omega}_{t}, \tag{16}$$
$$d\in\mathcal{D}_{\underline{III}},\quad \omega\in\Omega,\quad t\in[T_0,T].$$

where ϑ_{III} is the set of unstored resources. Presented in (16) functions of consumption of unstored resources $\varphi^{d\omega}_{z\alpha t}$ and $\varphi^{\kappa d\omega}_{s\alpha t}$ have the meaning and possess characteristics, si - milar to the presented in (15) relationships of utilizing stored resources.

Conditions of achiving production targets, depending on an opportunity of accumulating it over the design period, are introduced into the model in the form of (15) or (16). Besides, functions $\varphi^{d\omega}_{z\alpha t}$ have the meaning of a relationship of production targets amount of water resources quality and quantity.

Task Function

The WRS development over the design period is estimated by the cost function, including expences on construction and renovation, operation and maintenance, on output, delivery and purification of water as well as expenses, caused by a deviation of water quantity and quality from the optimum.

Functions describe a mathematical expectation of costs over the design period

$$f_z(b_z, x_z, y_z) = \sum_{z_\alpha \in Q_z} \Big[K_{z\alpha} b_{z\alpha} + \sum_{\omega \in \Omega} P^{\omega} \sum_{t \in [T_0, T]} [f^{\omega}_{z\alpha t}(x^{\omega}_{z\alpha t}, u^{\omega}_{z\alpha t}) + \\ + f^{\kappa\omega}_{z\alpha t}(x^{\omega}_{z\alpha t}, u^{\kappa\omega}_{z\alpha t})]\Big], \tag{17}$$

where coefficients $K_{z\alpha}$ reflect constant costs independent of operational regimes $x^{\omega}_{z\alpha t}$, $u^{\omega}_{z\alpha t}$. Functions $f^{\omega}_{z\alpha t}$ and $f^{\kappa\omega}_{z\alpha t}$ estimate variable costs dependent on $x^{\omega}_{z\alpha t}$ and $u^{\omega}_{z\alpha t}$. Components $f^{\kappa\omega}_{z\alpha t}$ don't equal 0 only for $z_\alpha \in S_{II}$. Functions $f^{\omega}_{z\alpha t}$ and $f^{\kappa\omega}_{z\alpha t}$ possess characteristics similar to those of functions $\varphi^{d\omega}_{z\alpha t}$ and $\varphi^{\kappa d\omega}_{s\alpha t}$, that is they are convex in each of the variables $x^{\omega}_{z\alpha t}$ and $u^{\omega}_{z\alpha t}$ and not convex in their totality [4] and $f^{\omega}_{z\alpha t}(0,0) = 0$.

Development Task

Determination of the optimum parameters and WRS regimes is described by the task A of determination of the optimum parameters and sources of the network Γ (J,S). The task deals with estimations of vectors z^0, x^c, y^0 with strategic components $z^0 = \{z^0_{z\alpha}, z_\alpha \in R\}$ and tactical components $x^c = \{x^{\omega c}_{z\alpha t}, z_\alpha \in R, \omega \in \Omega, t \in [T_0, T]\}$, $y^0 = \{y^{\omega c}_{z\alpha e t}, y^{\kappa\omega c}_{s\alpha e t}; z_\alpha \in R, s_\alpha \in S_{\overline{II}}, \omega \in \Omega, e \in L, t \in [T_0,T]\}$ which minimized the mathematical expectation of WRS costs

$$f(z,x,y) = \sum_{z \in R} f_z(z_z, x_z, y_z) = \sum_{z \in R} \sum_{z_\alpha \in a_z} \Big[K_{z\alpha} z_{z\alpha} + \sum_{\omega \in \Omega} P^\omega \sum_{t \in [T_0,T]} \big[f^\omega_{z\alpha t}(x^\omega_{z\alpha t}, u^\omega_{z\alpha t}) + f^{\kappa\omega}_{z\alpha t}(x^\omega_{z\alpha t}, u^{\kappa\omega}_{z\alpha t}) \big] \Big] \tag{18}$$

in the set G, formed by constraints (1)-(16). G is prescribed by the system of continuity equations of water and impurity flows

$$\sum_{i_\alpha \in a_i} \left[Q^\omega_{i\alpha, t+1} - k^\omega_{i\alpha t} Q^\omega_{i\alpha t} \right] = h_t \left[\sum_{s_\alpha \in S_i} k^\omega_{s\alpha t} q^\omega_{s\alpha, t-\theta} - \sum q^\omega_{s\alpha t} + b^\omega_{it} \right], \tag{19}$$

$$\sum_{i_\alpha \in a_i} \left[Y^\omega_{i\alpha e t+1} - \sum_{\gamma \in L} A^{\gamma e \omega}_{i\alpha t} Y^\omega_{i\alpha \gamma t} \right] = h_t \left[\sum_{s_\alpha \in S_i^+} y^{\kappa\omega}_{s\alpha e t} - \sum_{s_\alpha \in S_i^-} y^\omega_{s\alpha e t} + b^\omega_{iet} \right], \quad e \in L \tag{20}$$

by transformation equations of impurities in arcs, depicting river and canal reaches, users and sources without purification facilities in the network Γ (J,S).

$$y^{\kappa\omega}_{s\alpha e t} = \sum_{\gamma \in L} A^{\gamma e \omega}_{s\alpha t} y^\omega_{s\alpha \gamma, t-\theta} \tag{21}$$

by equality equations of impurity concentrations in arcs, outgoing from the same vertex

$$y^\omega_{s\alpha e t} q^\omega_{\tilde{s}\tilde{\alpha}, t} = y^\omega_{\tilde{s}\tilde{\alpha} e t} q^\omega_{s\alpha t}, \quad s_\alpha, \tilde{s}_\alpha \in S_i^-, \; e \in L \tag{22}$$

upper and lower limits of flow magnitudes in arcs and storage supplies as well as amounts and compositions of impurities in arcs and storages

$$z_{z\alpha} \underline{x}^\omega_{z\alpha t} \leq x^\omega_{z\alpha t} \leq z_{z\alpha} \bar{x}^\omega_{z\alpha t}, \quad z_\alpha \in R,$$

$$\sum_{e\in L_\nu} d^{w}_{z\alpha e t}\, z^{w}_{z\alpha e t} \le b_{z\alpha}\, \beta^{w}_{z\alpha t}\, x^{w}_{z\alpha t},\ z_\alpha\in R,\ \bigcup_\nu L_\nu = L,$$

$$\sum_{e\in L_\nu} d^{kw}_{s\alpha e t}\, y^{kw}_{s\alpha e t} \le b_{s\alpha}\, \beta^{w}_{s\alpha t}\, q^{kw}_{s\alpha t},\ s_\alpha\in S, \qquad (23)$$

$$z^{w}_{z\alpha e t} \ge b_{z\alpha}\, \underline{z}^{w}_{z\alpha e t},\ y^{kw}_{s\alpha e t} \ge b_{s\alpha}\, \underline{y}^{kw}_{s\alpha e t},\ z_\alpha\in R,\ s_\alpha\in S',$$

by restraints of the non-equality type caused by insufficiency of "non-water resources" and production targets of water consuming industries

$$\sum_{z_\alpha\in R} m^{d}_{z\alpha}\, b_{z\alpha} \le M^{d},\quad d\in D_{I},$$

$$\sum_{t\in[T_0,T]}\Big[\sum_{z_\alpha\in R}\varphi^{dw}_{z\alpha t}(x^{w}_{z\alpha t}, u^{dw}_{z\alpha t}) + \sum_{s_\alpha\in S_{II}}\varphi^{kdw}_{s\alpha t}(x^{w}_{s\alpha t}, u^{kdw}_{s\alpha t})\Big] \le M^{dw},\ d\in D_{II} \qquad (24)$$

$$\sum_{z_\alpha\in R}\varphi^{dw}_{z\alpha t}(x^{w}_{z\alpha t}, u^{dw}_{z\alpha t}) + \sum_{s_\alpha\in S_{II}}\varphi^{kdw}_{s\alpha t}(x^{w}_{s\alpha t}, u^{kdw}_{s\alpha t}) \le M^{dw}_{t},\ d\in D_{III}$$

by interaction conditions of characteristic functions of WRS elements' development versions and formation rules of impurity comlexes

$$b_{z\alpha}\in\{0;1\},\ \sum_{z_\alpha\in a_z} b_{z\alpha} = 1,\ z_\alpha\in R \qquad (25)$$

$$u^{dw}_{z\alpha t} = \sum_{e\in L}\delta^{dew}_{z\alpha t}\, y^{w}_{z\alpha e t},\ u^{kdw}_{s\alpha t} = \sum_{e\in L}\delta^{kdew}_{s\alpha t}\, y^{kw}_{z\alpha e t},\ z\in R,\ s_\alpha\in S_{II} \qquad (26)$$

and by initial conditions

$$q^{w}_{s\alpha t} = q^{w0}_{s\alpha t},\ y^{w}_{s\alpha e t} = y^{w0}_{s\alpha e t},\ s_\alpha\in S,\ t\in[T_0-\theta^{w}_{s\alpha t},\ T_0-1]$$

$$Q_{i\alpha 0} = Q^{w}_{i\alpha 0} = Q^{w}_{i\alpha T} = Q_{i\alpha T},\ Y_{i\alpha 0} = Y^{w}_{i\alpha 0} = Y^{w}_{i\alpha T} = Y_{i\alpha T},\ i_\alpha\in J \qquad (27)$$

In the task A the system of equations (19) is received from (12) by substitution of interconnection equations (2) of inlet and outlet arc flows and equations (5) for relationships of water losses from reservoirs and seas; coefficient of "storage gain": $K_{i\alpha t}^{w} = 1 - h_t \, \gamma_{i\alpha t}^{w}$. The system of equations (20) is received by substitution of equations (9) of impurity storage supplies in Γ (J,S) in (13).

The task A is a two-staged task of stochastic programming, in which first-stage strategic variables are characteristic functions of development varients of the system ξ ; second-stage tactical variables are vectors of water and impurity supplies Q and Y in storages and water flows q impurity flows y in arcs of Γ (J,S).

<u>Solution Task</u>

The task A is multiextreme, that is, it has local minima, different from the global one. Multiextremity of this task is caused by a non-convexity of the biseparable task function (18) and non-convexity of the permissible set G , assigned by bilinear constraints of the equation (22) type biseparable non-convex constraints of the equation (24) type and integer requirements(25) of strategical variables ξ . Specific features of the task A allow the method of its solution to be build.

The solution of the task A is reduced to selection of optimum vectors of a finite sequence of estimating convex tasks on the graph Γ (J,S), which are formed after the method, working out details of the diagram of branches and boundaries [16] . As a result a vector is obtained corresponding to the magnitude of the task function, different from the optimum one by not more than a prescribed value and exceeding the permissible set by not more than a prescribed error. This method is described in details in [16] .

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PHOSPHORUS IN BOTTOM SEDIMENTS OF CENTRAIN WATER BODIES AND ITS RELEASE FROM THE BOTTOM INTO THE WATER

M. V. Martynova and E. I. Kozlova

In estimating the role of bottom sediments in the phosphorus cycle in water body it is necessary to obtain quatitative relationships between the densities of phosphorus flows in the water-bottom system and their controlling factors. The absence of such investigations prevents a current understanding of the specifics of the cycle of matter and energy in an eutrophicating water body /1/.

We know very little of the transformation processes of phosphorus compounds in bottom sediments and of the release of phosphorus from bottom into the water, and this involves serious miscalculations in staging full-scale experiments. This main thing is the lack of a single methodological approach, the splitting up of the complex of interrelated problems into small fragments, thus preventing a phenomenon to be dealt with as an integral whole. This has resulted in a paradoxical situation when a tremendous volume of information cannot be practically put to an effective use.

The present work is an attempt to outline the complex of basic determinations that appear to be necessary in studying the processes of phosphorus transformations and its release from the bottom sediments, as well as to illustrate the effectiveness of the proposed approach.

The scheme of the determinations which were performed in the course of full-scale observations is shown in Fig. 1; the methods of chemical analysis used in sample processing are summarized in Table 1.

The interstitial solution was obtained not later than 3-4 hours after samplin, the silt being subject to centrifugation at 6 000 r.p.m. during 20 min and the solution being filtered through a membrane filter with 0.45 m dia. proses. Assessment of the organic matter destruction in bottom sediments and of the phosphorus flow from the bottom into the water was made in the experiments with undisturbed cores.

Table 1. Methods of chemical analyses

Index	Method of determination
O_2 dissolved in water	Winkler titration
P_{total} in water and interstitial solution	pershulphate oxidation
P_{min} in interstitial solution	Murphy-Raily
Phosphorus forms in the solid sedimentation phase:	
P_{total}	after Metha /4/
P_{min}	– " – –"–
$P_{Fe,Al}$ (nonapatite P)	extraction of 1 n NaOH during 16 hours at room temperature
P_{Ca} (apatite P)	extraction of 0.5 n HCI during 16 hours at room temperature
Number of microbenthos	direct count

The destruction of organic matter in bottom sediments due to aerobic processes was estimated from oxygen consumption; a total destruction of organic matter was determined from the release of carbon dioxide (variation of the HCO_3^- concentration in the water above the silt during exposition time). /2/

Investigations were carried out on two high-trophic lakes (Beloe and Chernoe), two mesotrophic lakes (Kubenskoe and Naroch) and two mesotrophic reservoirs (Uchinskoe and Pestovskoe) (Table 2).

Table 2. Certain characteristics of investigated sediments (0 - 2 cm silt layer)

Water reservoir	Sampling date	Sampling depth, m	Water temperature at bottom °C	O_2 consentration in in near - - bottom water, mg/l	Natural silt moisture, %% of silt weight	Content in sediments C_{org} in %% of dry silt weight	C_{carb}	Total number of bacteria, bil. cm³ of silt	Nature of sediments
Beloe lake									
samp. 1	03.09.84	6,2	12,2	8.2	92.0	15.4	-	2.0	fine-detrite silt
samp. 2	05.06.85	7.0	12.0	3.4	93.3	18.4	3.1	3.3	- " - - " -
samp. 3	12.08.85	9.0	13.0	not found	93.4	15.2	-	3.2	- " - - " -
samp. 4	12.06.85	4.5	16.0	7.9	85.7	15.0	2.2	1.3	- " - - " -
Chernoe lake									
samp. 1	28.08.84	2.2	15.0	9.0	94.5	24.0	1.1	0.9	coars-detrital silt
samp. 2	19.06.85	3.5	17.0	8.5	90.2	17.8	1.2	1.0	- " - - " -
samp. 3	26.08.85	3.0	22.0	8.5	94.5	20.3	-	2.5	- " - - " -
Naroch lake	09.09.85	17.0	16.6	9.2	92.5	23.5	-	-	fine-detrital
Kubenskoe lake	26.06.83	3.8	13.2	8.6	27.0	3.3	1.5	1.4	fine-aluerolitic silt
Pestovskoe reservoir	04.06.84	4.0	21.6	-	55.7	5.1	0.46	0.8	clayed silt
Uchinskoe reservoir	23.07.84	10.0	-	9.2	68.8	5.9	0.38	-	clayed silt

Sediments in three lakes (Beloe, Chernoe, Naroch) are represented by detrial silt, its organic matter being of planktonik-macrophytic origin, in lake Chernoe - with peat admixture. The content of organic matter in the bottom sediments of the above lakes, as determined from the loss in calcination, is from 30 % to 47 %. The sediments in the reservoirs are represented by calyed silt with 10 -12 % content of organic matter, while those in the Kubenskoe lake - by fine-aleurolitlic silt containing only 6.6 % or organic matter (Table 2).

The particularly mobile forms of phosphorus in bottom sediments are its dissolved compounds - organic and mineral, whose concentration in the interstitial solution from the water bodies in question varies from 0.15 mgP/l (Pestovskoe reservoir) to 14 mgP/l (lake Chernoe). Its magnitude is determined by the ratio of the input rate of the phosphorus compounds into the interstitial solution and the rate of their release from the solution. Organic compounds of phosphorus make their way into the interstitial solution because of the hydrolysis of organic matter and are released from it due to mineralization and diffusion or advection from the bottom into the water. Mineral compounds of phosphorus come into the interstitial solution as a result of organic matter mineralization and desorption from the surface of silt particles and are released from it either through sorption and diffusion or advection into the near-bottom water. The release rate of phosphorus in organic matter destruction in the investigated sediments from the water reservoirs was found to be varying from 0.8 to 2.7 mgP/m^{2}day, i.e. 3.5 times, whereas the concentration of phosphorus in the interstitial solution varies nearly a 100 times and the density of the phosphorus flow from the bottom - more than a 100 times. Consequently, the differences in the phosphorus concentration in the interstitial solution are determined not so much by the intensity of organic matter destruction in silts as by the differences in the sorptional-exchange processes and in the densities of diffusive and advective flows of the interstitial solution (the latter are produced in tetrital silts of lakes Beloe, Chernoe and Naroch as a result of gas release which in the lakes Beloe and Chernoe is far more intense than in lake Naroch). The sorptional-exchange processes are controlled by the redox potential and are dependent on the physicochemical properties of silts, a highly important factor determining the phosphorus concentration in the interstitial solution.

With respect to physico-chemical properties the investigated silts can be subdivided into two groups: those with a high (lakes Beloe, Chernoe, Naroch) and with a low (lake Kubenskoe and the two reservoirs) content of organic matter. Sediments in the first group are observed to be similar as regards the content of total, organic and apatite phosphorus in the solid phase, as well as the concentration of organic phosphorus in the interstitial solution.

As regards the content of mineral and nonapatite phosphorus in the solid phase of the silt and the concentration of phosphates in the interstitial solution, the silts of the Naroch lake are perceptibly different from those of the two other lakes, which are significantly higher with respect to the above characteristics.

It was previously shown /1/ that a significant part of nonapatite phosphorus in silts is of the secondary origin (resulting from organic matter mineralization in the silts and the subsequent sorption of phoshates). Apparently, in lake Naroch's sediments the processes of secondary formation of nonapatite phosphorus are proceeding at a slower rate than is the case with the silts in the two other lakes. This may be associated both with a low sorptional capacity of silts with respect to phosphate (an insignificant content of iron hydroxides in the silts) and with a relatively low mineralization of organic phosphorus in the silts, as a result of which little phosphates are formed. The relationship between the content of $P_{Fe,Al}$ per 100 g of silt of natural moisture and the concentration of phosphates in the interstitial solution indicates that the silts in lake Naroch are envolved with the sorptional capacity with respect to phosphates not lower than the silts in lakes Beloe and Chernoe. Consequently, the low content $P_{Fe,Al}$ in the topmost two-centimeter layer of the silts in lake Naroch is due to a relatively nonhigh intensity of organic phosphorus mineralization (data on organic matter destruction in Table 3 are the results of one-time determinations and reflect the intensity of the process soley for a specific moment of time).

The mineralization of organic compounds in the sediments of water bodies of low or average trophicity appears to be proceeding at a slower rate than is the case in eutrophic and hypertrophic water bodies. This inference is in agreement with the presence of a direct dependence of organic matter destruction in bottom sediments upon primary plankton produce, as was obtained for water bodies of different trophicity /1/.

A higher intensity of phosphate formation with the rising trophic level of a water bodies is behind the growing content of sorbed phosphates in the silts (nonapatite phosphorus) and a rise of the equilibrium concentration of dissolved phosphates. The fluctuation range of the phosphate concentration in the interstitial solution according to the redox potential of the medium becomes much greater. Correspondingly, the density of the phosphate flow from the bottom into the water under anaerobic conditions becomes greater (lake Beloe, samp. 3).

Clayey silts in the Pestovskoe and Uchinskoe reservoirs and aleurolitic silts in lake Kubenskoe are characterized by a low content of total, organic and apatite phosphorus (Table 3), which is associated with sedimentation specifics. Nor here the intensity of organic matter destruction in the silts corresponds to the phosphorus concentration in the interstitial solution and to the content of $P_{Fe,Al}$, which are determined by the physicochemical properties of the silts. Judging from the value of the $P_{Fe,Al}/P_{min}$ ratio, the maximum sorptional capacity with respect to phosphates is inherent in the silts of the Pestovskoe reservoir, the minimum capacity in those of the Uchinckoe reservoir, as corresponds to the phosphate concentratiob in the interstitial solution from the two water bodies (Table 3). Thus, although the organic matter destrction in the silts is the only source of mobile phosphate, its effects on their content (in the form of nonapatite phosphorus of autogenous origin) becomes evident over a long period of the time (year, a number of years). The concentration of phosphates in the interstitial solution and the relationship between their dissolved and sorbed forms at every moment of time are determined by the sorptional capacity of the silts.

Table 3. Phosphorus forms in bottom sediments; transformation and removal processes

Water reservoir	Content in bottom sediments									$\frac{C_{org}}{P_{org}}$	Organic matter destruction in silts				Phosphorus flow from bottom, mg P/m²da	$*\frac{P_{Fe,Al}}{P_{min}}$
	C_{org}, % abs. dry silt	Interstitial of solution, mg P/l			Solid phase of silt, mgP/100 g dry silt						mg C/(m²day)		mgP/(m²day)			
		P_{tot}	P_{org}	P_{min}	P_{tot}	P_{org}	P_{min}	$P_{Fe,Al}$	P_{Ca}		aerobic	total	aerobic	total		
Lake Beloe																
samp. 1	15.4	2.50	0.03	2.47	215	95	120	47	60	162	170	270	1.1	1.7	12.8	1.5
samp. 2	18.4	1.54	0.07	1.47	271	131	140	68	44	140	117	167	0.8	1.2	10.0	3.0
samp. 3	15.2	5.40	1.42	3.98	276	145	131	62	46	105	none	176	none	1.7	82.0	1.0
samp. 4	15.0	0.58	0.23	0.35	182	82	100	36	41	183	98	143	0.5	0.8	2.2	12.7
Lake Chernoe																
samp. 1	24.0	0.59	none	0.59	242	67	175	121	36	358	115	395	0.3	1.1	1.1	11.2
samp. 2	17.8	2.30	none	2.30	345	210	135	70	39	85.5	143	227	1.7	2.7	47.6	3.0
samp. 3	20.3	14.0	0.40	3.6	213	104	109	64	31	195	246	410	1.3	2.1	44.3	0.3
Lake Naroch	23.5	0.37	0.27	0.10	196	139	57	16	36	169	135	225	0.8	1.3	1.8	12.0
Lake Kubenskoe	2.6	0.28	0.03	0.25	32	14	18	4	14	185	190	320	1.0	1.7	1.2	11.6
Pestovskoe reservoir	5.1	0.15	0.04	0.11	117	30	87	59	10	170	135	225	0.8	1.3	0.6	26.0
Uchinskoe reservoir	5.9	0.75	0.21	0.54	108	39	69	44	8	151	87	145	0.6	1.0	-	2.6

Note: $P_{Fe,Al}$ - in mg/100 g silt of natural moisture; P_{min} - in mgP/l.

An important specific of lakes with deterital silt is gas release from the bottom, which is particularly intense in samp. 3 of lakes Beloe and Chernoe. To form an idea about the effect of gas release on the stratification of phosphorus forms in the silts, we carried out layer-by-layer investigations of the sediments in lakes Beloe, Chernoe and Naroch.

Stratification of phosphorus forms in the solid phase of sediments is an integral index of the specifics of their long-term accumulation and transformation, which is not subject to yearly fluctuations (exept for the 0 - 2 cm layer). Contrary to this, the stratification of phosphorus forms in the interstitial solution depends on the season /1/ and is determined by the activity of the microflora controlling the formation of not only dissolved phosphorus forms but also that of the gases. Gas release, as noted during the exposition of silt-filled tubes from gas bubbles in all of the layers, does not bring about a vertical equalization of phosphorus concentrations in the interstitial solution, while improverishing the downmost silt layers. It appears to be particularly active in the 10 - 15 mm layer. Here forms the main flow of gas which, rising up to the bottom surface through silt layers, is gradually attenuating. Therefore, part of phosphorus carried out with the flow from the downmost silt layers will accumulate in the topmost layer even in the absence of an oxidized silt film at the surface. This can be distinctly observed in the case of an intensive gas release (lakes Beloe and Chernoe). The gas flow and the amount of phosphates brought out of and into different silt layers probably varies from season to season. Following up the effect of this on the distribution of $P_{Fe,Al}$ in the silts, by allowing for possible inhomogenity of inflow into the deposits in different years proves to be a very complex thing. It is hardly to be doubted, however, that this has an equalizing effect on $P_{Fe,Al}$ stratification in the deposits of lakes Beloe and Chernoe. In the silts of lake Naroch this flow is weak and is not observed to have a perceptible effect on the vertical distribution of $P_{Fe,Al}$ (Table 4). A continuous increase of the content of this phosphorus from into the depth of the sediments is the result of P_{org} mineralization and of the sorption of the bulk of the phosphates formed.

A comparison of the data characterizing the intensity of the phosphorus release during the mineralization of organic matter in silts (as determined from C/P ratio in the organic matter of the silts and a total destruction) and the experimentally determined phosphorus flow from the bottom enables us to assess certain mechanisms of phosphorus transformation and phosphorus release from bottom sediments. In lake Kubenskoe and Pestovskoe reservoir the principal mechanism of phosphorus release was found to be a concentrational diffusion of compounds release in organic matter destruction at the continuously renovating bottom surface. The phosphorus flow from the bottom is less than the intensity of phosphorus release in the destruction of organic matter (Table 3). This gives an oppurtunity of estimating the amount of sorbed phosphates: 0.5 and 0.7 mgP/m^2day for the two water reservoirs, i.e. the sorptional capacity of silts in the Pestovskoe reservoir is greater as that of the silts in lake Kubeskoe (Table 3). In the case of detrital silts the phosphorus flow from the bottom exeeds the quantity of phosphorus released in organic matter destruction on account of an advective movement of the interstitial solution resulting from gas release. The total flow of phosphorus from the bottom ($P\Sigma$) can in this case be represented in the following manner:

$$P_{\Sigma} = P_{g} + P_{m} + P_{d} - P_{s}, \text{ where} \qquad (1)$$

P_{g} is phosphorus released from the sediments with an advective flow arising in gas release; P_{m} is phosphorus liberated in the mineralization of organic matter; Pd is phosphorus being released as a result of concentration diffusion from the deeper-laying silt layers; P_{s} is sorbed phosphorus, Hence

$$P_{g} = P_{\Sigma} - P_{m} - P_{d} + P_{s} \qquad (2)$$

P_{Σ} was determined in aquarium experiments on phorphorus release from bottom sediments; P_{m} was estimated from the total destruction of organic matter in silts, as determined experimentally, and from the C/P ratio in the organic matter of these silts; P_d was calculated from the equation

$$P_d = D \cdot \Delta C : 1, \text{ where} \qquad (3)$$

D is the phosphorus diffusion coefficient in silts, assumed to be equal to $1 \cdot 10^{-6}$ cm^2/sec/1/; ΔC is a difference between the phosphate concentrations in the interstitial solution (uppermost two-centimeter layer) and in the near-bottom water, as determined experimentally; 1 is the pathway length equal 1 cm.

Table 4. Stratification of phosphorus forms in the silt of three lakes.

Sampling site	Silt layer,	Interstitial solution, mg P/l			Solid phase, mg P/100 g dry silt					C_{org}/P_{org}
		P_{tot}	P_{min}	P_{org}	P_{tot}	P_{org}	P_{min}	$P_{Fe,Al}$	P_{Ca}	
	0-2	5.40	3.98	1.42	276	145	131	62	43	105
	2-5	3.70	3.25	0.45	295	164	131	64	46	97
Lake Beloe	5-10	2.57	2.15	0.42	255	125	130	61	48	122
samp. 3	10-15	1.63	1.43	0.20	266	138	128	68	48	145
	0-2	14.4	13.6	0.4	213	104	109	64	31	195
	2-5	11.0	10.6	0.4	179	85	94	57	31	214
Lake Chernoe	5-10	6.7	6.0	0.7	196	96	100	54	34	195
samp. 3	10-15	1.7	1.6	0.1	493	133	360	268	64	129
	0-2	0.37	0.10	0.27	196	139	57	16	36	169
	2-5	0.52	0.05	0.47	180	88	92	28	39	284
Lake Naroch	5-10	0.59	0.06	0.53	200	112	88	40	29	223
	10-15	0.29	0.08	0.21	185	155	130	78	49	373

To estimate the value of P_m we carried out an experiment for determining the phosphorus flow from the bottom in the presence of mercuric chloride, 2 ml of the saturated solution of the latter being depostited on the silt surface. After 10 - 15 min the silt was flooded with filtered near-bottom water. The experiments were made after the aforementioned procedure parallel with the other ones. Deposition of mercuric chloride on the silt surface was followed by the appearance of an ochrous film 6 to 11 mm thick (Table 5) due to microlora inhibition and silt oxidation. The phosphorus flow from the bottom, as determined in the given case, $P_{\Sigma mc}$, can be expanded in the following manner:

$$P_{\Sigma mc} = P_{si} + P_{cl} - P_{sm},\ \text{where} \qquad (4)$$

P_{sm} is the phosphate sorption by the silt in the experiment with mercuric chloride. Hence

$$P_{si} = P_{\Sigma mc} - P_{cl} + P_{sm} \qquad (5)$$

Tence from (2) $P_{\Sigma} - P_{m} - P_{cl} + P_{s} = P_{\Sigma mc} - P_{cl} + P_{sm}$ (6)

$$\text{and } P_{\Sigma} - P_{\Sigma mc} - P_{m} = P_{sm} - P_{s} \qquad (7)$$

Assuming in the first approximation that phosphates are sorbed by the ochrous film of the silt alone and their sorption is proportional to the thickness of the film, we shall determine the thickness of the film as corresponding to $P_{sm} - P_{s}$ (a difference between the thickness of oxidized film in the experiments with mercuric chloride and without mercuric chloride) and calculate how much phosphates are sorbed by the oxidized (ochrous) film of the silt 1 mm thick in each case (Table 5).

Table 5. Estimating phosphate sorption by silts

Sampling site	Thickness of oxidized silt layer, mm, in experiments: without mercuric chloride	with mercuric chloride	$a_1 - a_2$	$P_{sm} - P_{s}$, $\frac{mg\ P}{m^2 \cdot day}$	Phosphate sorption by 1 mm silt layer $\frac{mg\ P}{m^2 \cdot day}$
Lake Beloe					
samp. 2	0.5	11	10	3	0.3
samp. 3	none	none	none	none	none
samp. 4	1.0	6	5	1	0.2
Lake Chernoe					
samp. 2	0.5	11	10	72	7
samp. 3	0.5	6	9	46	9

Taking into account the thickness of the ochrous film in the experiments without mercuric chloride, we shall estimate the quantity of sorbed phosphates. As a result, we have obtained tentative values of all constituents in the equation (1) from which we can estimate the value of P_{g} (Table 6).

Table 6. Principal constituents of the phosphorus flow from the bottom, mg P/m^{2}day

Sampling site	P_{Σ}	P_m	P_{d}	P_{w}	Pg mg/P(m2day)	Pg %%of P_{Σ}
Lake Beloe						
samp. 2	10	1	1	0.2	8	80
samp. 3	82	2	3	none	77	94
samp. 4	2.2	0.8	0.2	0.2	1.4	64
Lake Chernoe						
samp. 2	48	3	1	4	48	100
samp. 3	44	2	12	5	35	80

As seen from the Table, the phosphorus flopw produced in gas release from the silts in lakes Beloe and Chernoe makes up from 64 % to 100 % of total removal of dissolved phosphorus from the bottom. Its significance in either lake is observed to increase with greater depth.

No experiments for determining the phosphate sorption by the silts in lake Naroch were undertaken; however, proceeding from the date in Tabe 3 we may assert thath the advective flow set up by gas release exeeds 30 % of total phosphorus flow from the bottom.

The process of gas formation in the bottom sediments of lakes is a fairly widespread phenomenon /3/.

Correspondingly, special attention should be given to this mechanism of phosphorus release from bottom sediments of not only high-trophic but also mesotrophic lakes with a high content of organic matter in the silts. Possibly, precisely this mechanism of phosphorus release is responsible for a slow deeutrophication of a number of lakes (Norwiken, Södra Bergundasjön, etc.), which becames effective solely after the removal of the bottom sediments layer from where gaas release is taking place (lake Trummen). Estimating the significance of this flow in each separate case and of its determining factors (gas release intensity, phosphate concentration in the interstitial solution, etc.) in low-trophic lakes may contribute to determining their resistance with respect to eutrophication, and in the case of hupertrophic lakes - to identify the sites and sediments depth to be removed for the purpose of quick deeutrophication.

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THE EFFECT OF THE FORMS OF HEAVY METALS AND THEIR MIGRATION IN WATER-COURSE BOTTOM SEDIMENT ON SECONDARY POLLUTION

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An insight into the interaction at the water - bottom interface and the adjacent boundary layers is indispensible for evaluating the quality of water in bodies of water and water-courses and for improvement of its forecasts. The rates of compound exchange, transfer, and transformation in bottom sediments are influenced by numerous factors. This paper will discuss the ionic exchange, a mechanism which makes the strongest quantitative impact on the matter wash out, with an allowance for the forms of heavy metals both in a porous solution and in the solid skeleton of the soil. More specifically, the research was carried out in bottom sediments of a city river polluted by waste and surface waters.

The rate of formation and the chemical composition of such sediments depends primarily on the soil, the hydrological conditions, the kind of substance, migration and the specifics of the biochemical migration cycles in water landscapes.

The solid drift is supplied above all by the hydrographic network; medium and large rivers transport it while washing away and accumulating the sediments. The waste drift is an integral characteristic of the pollutants arriving in the river network with the surface discharge which washes away the soil cover, the bottom, and banks. The ratio of these two groups differs for tributariss of different orders and are hard to distinguish. In city rivers the balance is tipped strongly in favor of the drift arriving in the river network with the surface drain from residential areas. For river waters the data on solid discharge are summarized as charts of solid waste magnitudes. Thus for the Volga drainage area the mineralogical content of the suspension is, in percent of the total: quartz, 10; field spar, 3; and clay minerals, 37. The average composition of the clay is, in percent of the total: illit, 44; chlorit, 20; montmorillonit, 36 /3/. The average suspension composition is, in $n \times 10^{-4}$ percent: Ba, 350; Co, 11; Cs, 183 Cu, 86; Ni, 76; Zr, 76; Zn, 260; and V, 102. For our purposes the most interesting is the form of heavy metals in the suspension.

In the course of hypergenesis the clay minerals of soils and the waste mantle adsorb metals, metal oxides, and organic substances on their surfaces. Arriving in the river network and accumulating in bottom sediments these compounds make a significant contribution to biogeochemical migration and can under certain conditions become proces solutions and diffuse into the river waters.

In studying the kind of compounds which included iron, copper, and zink in the suspension and in the solid skeleton of bottom sediments the compounds different in mobility were successively leached by extracts. To identify surface-adsorbed forms of metals extracts, 1M solution of acetic acid was used with pH = = 2.3 [11]. This extract partly acted on amorphous iron and managanese hydroxides and metals adsorbed by them. From fresh amorphous hydroxides of manganese and iron 0.05 percent of the former and 1.8 percent of the latter were extracted and 20 to 50 percent of metals adsorbed by iron hydroxides were extracted.

Once the organic matter destroyed five times by hydrogen peroxide extraction by 1 M acetic acid makes it possible to estimate the fraction of metal which is attached to different forms of organic matter in bottom sediments.

Metal compounds bound in amorphous iron and manganese hydroxides were extracted by the Chester reactant (1 M of NH_4Cl in 25 percent solution of acetic acid). This extraction does not affect more stable decrystallized iron and mangenese hydroxides [12].

Extraction by hydrochlorinated alcohol dissolves decrystallized iron and manganese hydroxides and the metals attached to in them [1]. Metals attached to the crystalline structure of alumosilicates and clay minerals were identified following decomposition in the HF - H_2SO_4 - HCl mixture.

These extracts do not practically affect the crystalline structure of clay minerals.

Today there is no generally applicable way to extract some forms of metals without affecting to some degree the others. For this reason the proposed approach can only estimate what fraction of metals can fairly easily move into a porous solution of bottom sediments and then into river and lake waters.

The data in Table 1 on the phase composition of iron, copper, and zink in the suspended matter suggest that a major amount of metals migrates in the hard drift as silicate forms and iron and manganese adsorbed on hydroxides which make crusts on the fragments and clay pieces. Accumulating in the river beds the suspended matter turn into alluvial sediment sooner or later, depending on the size of particle. Suspended substances which arrive in the surface flow from urban areas are usually enriched with heavy metals. The drift of metal-rich sewage also enrich the suspensions. Metal-enriched scattered flows of complex genesis which form in the alluvial sedimentation may be resposible for secondary pollution of river waters.

Phase analysis of river bottom sediments leads to the following conclusions, Table 1:

1. More than one half of the total amount of iron in bottom sediments is mobile. Much of it are adsorbed forms which easily find themselves in porous solution where the associated metals are in short supply. A relatively small amount of iron is attached to organic matter and amorphous hydroxides whereas much of the total amount of iron is attached to decrystallized oxides. Iron adsorbed by the solid skelton of the ground and in organic matter and amorphous hydroxies can enrich the porous solution and is potentially dangerous for river waters.

Much of the total amount of zink and copper in bottom sediments is mobile in adsorbed forms and metals attached to organic matter and amorphous iron and mangenese hydroxides. Consequently, much of these zink and copper can move into the porous solution and river water from bottom sediments with both the water-soluble phase of the pollutant in the porous solution and adsorbed on the drift particles in which the suspension carrying flux and its bed continuously exchange.

As a result, the amount of pollutant arriving in the river water from bottom sediments is expressed as

$$N = N_w + N_s \qquad (1)$$

where N_w and N_s are the pollutant phases, water-solved and adsorbed on the drift particles.

Chemical studies of the porous solutions of bottom sediments suggest that

(1) The values of concentrations of different metals in porous waters of bottom sediments in a small river flowing through a city change very significantly along its course. The metal

content increases, as a rule, downstream of the mouths of more polluted tributaries and sewage outlets, Table 2.

(2) Humin and fulvic acids account for 92 percent of the total organic matter, the content of the former ranging in the suspensions from 10 to 70 μg C/l and in solutions, from 107 to 450 μg. The fulvic-acid content in solution was 2380-3085 μg C/l. Metals make complex compounds with fulvic and, to an extent, humin acids. In this way copper, iron, zink, and other elements increase their migrating capacity under any redox conditions. The solubility of metals in an excess of fulvic acid increases because of the high hydrating capacity of fulvic acid molecules.

(3) In solutions 88 to 93 percent of metals are in complex compounds with fulvic acids and four to six percent with humic acids. There are practically no ionic forms or forms with inorganic ligands. Some amount of metals with colloidal organic matter can be adsorbed on polar iron hydroxides and clay particles.

(4) Humic, fulvic-, and low molecular organic acids dissolve amorphous iron hydroxes well (the kinetics of this process was not studied) to form complex metal compounds with humic and fulvic acids which are easily dissolved in water.

The flow of the water-solved pollutant phase from the water thickness depends on both molecular and turbulent diffusion (because the boundary layer in the suspension carrying flux is not laminar [4]), the effect of the flux below the river bed and its interaction with that in the river bed [10], the life activity of the semi-submerged water vegetation [6], etc.

It is important that the ratio of N_W and N_S is influenced by the kinetics of pollutant adsorption and desorption on drift particles, a phenomenon that has not been profoundly explored [7].

Turbulent diffusion affects the flow of the water-soluble pollutant phase from the bottom by chaning the rate of pollutant transfer from the near-bottom zone of the flow into its main-stream or by influencing the boundary conditions of the pollutant diffusion problem at the water - bottom interface. In the case under discussion this leads to a rough estimate of the scale of secondary river pollution by metal-polluted bottom areas; one of the boundary conditions is

$$C(z,t)_{z_0=0} = C_B = const \qquad (2)$$

where z_0 is the mark of the bottom surface (the z axis being downward-bound);

C_B is the pollutant content in the water flow.

Let us have a closer look at the formation of an upward flow of pollutants resulting from molecular diffusion described by the equation

$$\frac{\partial c(z,t)}{\partial t} = D \frac{\partial^2 c(z,t)}{\partial z^2} \qquad (3)$$

where $C(z,t)$ is the pollutant concentration in the porous solution of the bottom at time t in a point whose vertical coordinate is z ;

D is the diffusion coefficient of the pollutant.

The mass m of pollutants moving from the bottom into the water is usually determined by the first Fick law of diffusion

$$\frac{\partial m}{\partial t} = DF \frac{\partial c}{\partial z} \qquad (4)$$

where F is the polluted bottom area.

The pollutant content gradient in the latter equation is determined, as a rule, by equation (3) and depends on the boundary and initial conditions of this equation.

Let us take up a very simple case where the bottom soil thickness is assumed infinite; at the initial time the pollutant content C_r in the porous solution is constant along the depth while the content at the water - bottom boundary (with $z = z_0$) is constant and equal to that in the water. In other words, equation (3) is to be solved with an initial condition

$$c(z_0 \leq z \leq \infty, t = 0) = C_r$$

and boundary conditions

$$c(z = z_0; t > 0) = C_B$$

$$\left.\frac{\partial c}{\partial z}\right|_{z \to \infty, t \geq 0} \to 0$$

In this case the pollutant content gradient depends on z and t in the following way [9]

$$\left.\frac{\partial c}{\partial z}\right|_t = \frac{C_r - C_B}{\sqrt{\pi D t}} \exp\left(-\frac{z^2}{4Dt}\right)$$

and at the bottom

$$\left.\frac{\partial c}{\partial z}\right|_{z = z_0} = \frac{C_r - C_B}{\sqrt{\pi D t}} \tag{5}$$

Integration of equation (4) with boundary conditions (5) yields the pollutant mass arriving during a time interval from a bottom area equal to

$$m = 2F(C_r - C_B)\sqrt{\frac{D\tau}{\pi}} \tag{6}$$

In the case of a finite thickness (h) the solution of (3) analogous with the solution (6) takes the form [5]

$$m = (C_r - C_B) h F \left[1 - \frac{8}{\pi^2} \varphi(\alpha)\right] \tag{7}$$

where $$\varphi(\alpha)=\sum_{n=1}^{\infty}\frac{1}{(2n-1)^2}\exp[-\pi(2n-1)^2\alpha],$$
$$\alpha=\pi DT/4h^2, \qquad (8)$$

T being the time interval from the onset of diffusion to a specified instance.

If the condition

$$\sqrt{DT}<<h \quad \text{, i.e. } \alpha\ll 1 \qquad (9)$$

holds, fairly cumbersome formula (7) can be replaced by (6). Then the value of would be somewhat exaggerated. The relative error of replacing equation (7) by (6) is estimated as [9]

$$2\exp\left(-\frac{h^2}{DT}\right). \qquad (10)$$

The processes occurring in bottom sediments (such as transfer of pollutants adsorbed on the solid skeleton of the bottom soild into the porous solution) may also change the value of C_r. This phenomenon is described by a diffusion equation with a source

$$\frac{\partial c}{\partial z}=D\frac{\partial^2 c}{\partial z^2}+kc \qquad (11)$$

where K is the constant of substance transformation.

The mass of the substance moving from polluted bottom sediments with the initial and boundary conditions of equation (11) having the form

$$c(z_0\leq z\leq\infty,\ t=0)=C_r$$
$$c(z_0=z;\ t>0)=0$$

has been obtained in Ref. [8] as

$$m=C_rF\sqrt{\frac{D}{\pi}}\left(2\sqrt{t}+\frac{2}{3}k\sqrt{t^3}+\frac{1}{5}\sqrt{t^5}\right).$$

When the washing of pollutants from porous waters of bottom sediments is believed to be offset by the transformation of these substances from the solid skeleton into the liquid phase the content gradient $\frac{\partial c}{\partial z}$ at the water - bottom interface may be made constant

$$\left.\frac{\partial c}{\partial z}\right|_{z=z_0} = \lim_{\Delta z \to 0} \frac{c(z_0+\frac{1}{2}\Delta z)-c(z_0-\frac{1}{2}\Delta z)}{\Delta z} = const \approx \frac{C_r - C_B}{\beta d} \qquad (12)$$

where d is the average diameter of bottom soil particles and β is a factor.

The estimate of the metal flow from bottom sediment into the water by formula (6) is unduly low because it neglects the possible movement of metals into the porous solution from the solid skeleton in the case of a depleting solution.

With our today's knowledge of the metal adsorption and desorption rates by river drift particles we can only estimate the upper and lower bounds of the amounts of pollutants which can move into the river bed flow from the bottom sediment. The upper bound of the water-soluble metal phase is determined from the relation (4) with a boundary condition (12) whereby the rate of metal washing from the porous solution would not exceed the rate at which these pollutants move from the solid skeleton into the porous waters. By integrating (4) we see in this case that the mass of a certain metal which has left over the time interval t a bottom area equal to F is

$$m_\Delta = DF \frac{C_r - C_B}{\beta d} t. \qquad (13)$$

Leaving out accumulation of pollutants by the bottom sediment (with their flow from the bottom being negative), the lower-bound may be nearly zero when, for instance, a geochemical barrier on the interface prevents the wash out of metals.

This case is not interesting for analysis; therefore the conditional lower bound will be the mass (m_0) of metals from the bottom sediment whose porous waters do not receive metals from the solid skeleton. This mass is

$$m_0 = 2F\left(C_r - C_B\right)\sqrt{\frac{Dt}{\pi}}$$

The ratio of the bounds is independent of the river bed because it follows from the latter equation and (13) that

$$\frac{m_\Delta}{m_0} = \frac{\sqrt{\pi D t}}{2\beta t} = \gamma\sqrt{t} \qquad (14)$$

The value of $m_\Delta m_0^{-1}$ is proportional to $\sqrt{t}$. Let us estimate the order of magnitude of the proportionality coefficient γ of this relation. For a rough estimate of the coefficient of metal diffusion from fresh water slimy and sandy soils one can assume, Ref. [6], that

$$D \approx n\, 10^{-6}\, cm \times s^{-1};\ n \sim 1 - 10.$$

In estimating the metal concentration gradient in a porous solution in the vicinity of the water - bottom sediment interface the concentration changes from C_r to C_w in a stratum whose thickness is equal to that of the under-channel bed flow which is initiated by the mainstream. This thickness has been found experimentally [2, 8] to amount to four to six diameters of particles, i.e. β = 4 - 6. The average diameter is about 50 um. Consequently,

$$\gamma = \frac{\sqrt{3.14(1-10)\times 10^{-6}\, cm^2\, s^{-1}}}{2\,(4-6)\times 5\times 10^{-3}\, cm} \sim 10^{-1}\, s^{1/2}$$

and

$$\frac{m_\Delta}{m_0} \simeq 10^{-1}\sqrt{t}. \qquad (15)$$

The resultant estimate shows, Fig. 1. that with t = 8.64 x x 10^4 s (one day) the ratio $m_\Delta m_0^{-1}$ amounts to about 30 while with t = 3.15 x 10^7 s (one year), to 500. The estimate (15) is inapplicable with small values of time intervals t because the content gradient in a point is replaced by the difference of concentrations at the boundaries of a nonzero stratum whose thickness is $\Delta z \sim \beta d$ (12); for this reason the plot of (15) at low t is a dotted line going into the point $m_\Delta m^{-1} = 1$ as t $\longrightarrow$ 0; this is sound physically.

Consequently, the way to state the boundary condition depends on the form of heavy metal in the bottom sediments and their mobility and makes a very significant impact on the estimate of secondary pollution of the river bed flow. This is why researchers of secondary pollution of bodies of water and watercourses should give special attention to the form of heavy metal in the bottom sediment and to the kinetics of pollutant desorption and adsorption on drift particles.

Fig. 1. Estimated ratio of metal masses arriving in the river from the bottom sediment under various conditions at the water - bottom interface: m_0 , metals do not move into the porous sc... on, or the boundary condition (5) holds; m_Δ , metals move into the porous solution from the solid skeleton of the bottom sediment at a rate ensuring truth of the boundary condition (12)

1. - t, s ; 2.- s ; 3. - minute; 4.- hour; 5.- day; 6.- week; 7.- month; 8.- year.

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Table 1

Phase composition of some elements in the suspensions and bottom sediments of small city rivers (in percent of the total)

Sampling site, km from the mouth	Element	Metal content in extractions, kind of compound				
		surface-adsorbed	organic	iron and manganese hydroxides		silicate
				amorphous	decrystallized	
Suspension of a river not subjected to anthropogenic effects	Fe	–	1	7	45	47
	Cu	2	4	43	37	14
	Zn	3	2	37	26	32
Bottom sediment of a small city river, 0	Fe	15	6	7	26	46
	Cu	88	3	2	2	5
	Zn	63	9	5	11	12
Bottom sediment of a city river, 4	Fe	43	5	11	17	24
	Zn	60	5	4	27	4
Bottom sediment of a small city river, 7	Fe	13	9	5	35	38
	Cu	89	3	2	3	3
	Zn	61	15	12	10	2
Bottom sediment of a small city river, 29	Fe	24	6	8	16	46
	Zn	69	2	1	18	10
Bottom sediment of a tributary of a small city river, 1	Fe	34	5	12	11	38
	Zn	79	3	3	7	8

Table 2

Metal content variations in porous waters of the bottom sedimal in a small rural river (contents with respect to the background content $\langle c \rangle$; benchmarks enumerated from the river mouth)

Bench-mark No.	Dimensionlees content of $C = \frac{c}{\langle c \rangle}$							
	Cu	Zn	Pb	Cd	Ni	Fe	Mn	C
1.	0.52	0.75	0.55	0.58	1.02	0.53	0.47	0.58
2.	1.71	0.18	1.57	1.17	5.42	2.03	1.11	1.64
3.	0.42	0.66	0.55	0.58	0.68	0.48	0.39	0.53
4.	2.25	2.88	1.97	3.50	2.88	1.14	1.00	2.37
5.	2.29	1.96	1.77	2.58	2.54	1.14	0.96	2.16
6.	0.82	0.60	0.55	0.83	0.42	0.46	0.62	0.53
7.	1.29	1.22	1.38	3.42	1.02	1.14	1.23	1.21
8.	1.11	1.09	1.34	1.42	0.85	1.00	1.22	0.63
9.	0.99	1.22	0.87	1.17	1.95	7.27	5.65	1.21
10.	1.89	1.67	1.61	0.67	0.51	1.02	1.05	1.08
11.	1.65	1.67	1.57	1.08	0.85	1.60	2.49	1.06
12.	-	-	-	-	-	-	-	-
13.	0.22	0.92	0.24	0.21	0.42	1.47	1.94	0.53

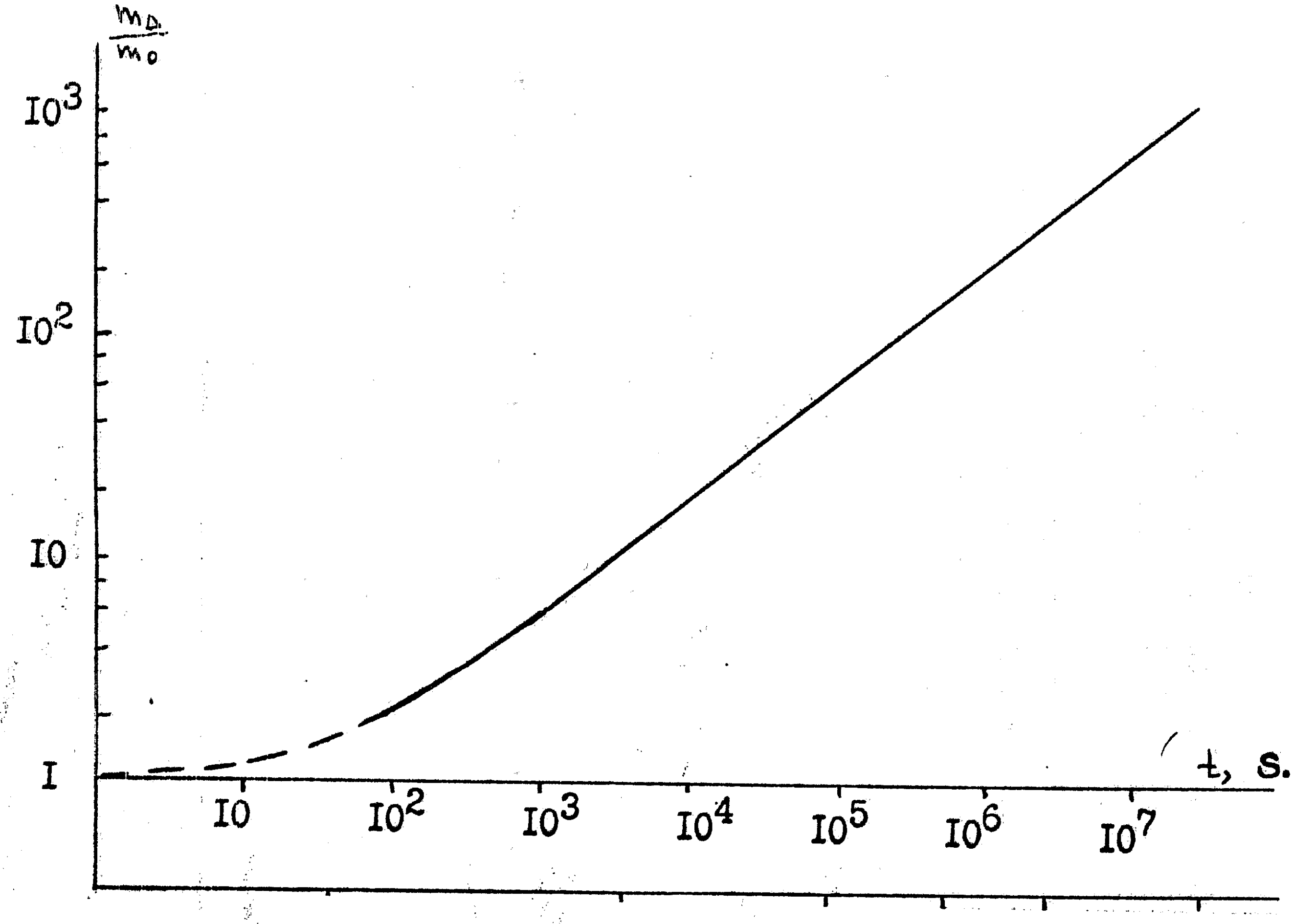

Fig. 1. Estimating masses of metals flowing into the stream from bottom sediments under various conditions on the water-bottom interface:

a) m_0 - the absence of metal inflow to the porous solution ensures boundary condition (5);

b) m_Δ - metal inflow to the porous solution from the bottom sediment at the speed ensuring boundary condition (12).

METHODOLOGY OF ESTIMATING THE EFFICIENCY OF INDUSTRIAL WATER RECIRCULATION SYSTEMS TO PREVENT NATURAL WATER POLLUTION

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The relationship between man and the environment becomes increasingly diverse, complex and voluminous. The progress of industrial and agricultural production inevitably leads to the human negative effect on nature, including the growing volume of wastes contaminating the atmosphere, soil and water sources. Wasteless technology is a radical method for preventing environmental pollution. However, development of wasteless industries is a complex technological and economic problem. Its complete solution requires time for industrial overhaul and substantial expenditures. Today the first and highly important step to sharply decrease the pollution of water sources, along with an appreciable cutback in water consumption,is introduction of water recirculation systems. It is known that waste water is a particularly multitonnage complex production waste. Disposal of even treated industrial waste water into water bodies and water courses results in water-quality deterioration and often in biological unbalance in surface waters.

Using today's water treatment methods, waste water needs to be further diluted with such quantities of pure water that cannot be obtained even from big rivers. For instance, in oil-processing industry the dilution frequency of treated waste water is up to 60, in pulp and paper industry - 20-40, in synthetic fibre production - 10-15, in synthetic rubber production - up to 2000, and in skin and leather industry - about 20 times /6/.

Furthermore, in waste water dilution, background pollution is always observed in water objects. Taking into account self-purification process in surface waters, we may assume the required degree of treated waste water dilution in major water courses to be at least 10 times on the average.

As for small rivers, particularly those flowing in the areas of major industrial centres, their runoff downstream is often comparable with the volume of inflowing waste water. Under these conditions, the pollutant concentrations in treated waste water are expected to meet the requirements corresponding to maximum permissible concentration standards for water bodies involving a specific type of water use. Achievement of a higher degree of treatment is not to be expected unless use is made of additional treatment processes, requiring large capital and operational expences.

Considering what has been said above, we may say for sure that the particularly effective water-protective measure both ecologically and economically are the use of water recirculation systems. The economic feasibility of recirculation systems stems from, among other things, the lower cost of the treatment of recirculated water as compared to that of waste water discharged directly into water bodies.

For instance, in automobile industry, a recirculation system may admit water with an oil product content of 15-30 mg/l (the cost of water treatment in this case is 3-5 kopecks/m^3), while in disposal to a water course or body this concentration must be reduced to 3-5 mg/l to achieve a 60-100-time dilution to meet the fishery maximum permissible concentration standard, the cost of water treatment rises, in this case, up to 30 kopecks/m^3 /5/. The biochemical oxygen demand in the recirculated water of cooling systems can be much larger than that of water discharged into a water body used for fishing purposes: 20-25 mg O_2/l at a maximum permissible concentration of 3 mg O_2/l. The same also true of total iron, copper, zinc, cyanides and synthetic surfacants whose concentrations in recirculated water and in water bodies following their dilution must be 4 and 0.05; 2.5-4 and 0.01; 10 and 0.05; 15 and 0.1 mg/l, respectively, /5/.

In some industries (electronics and radio engineering) technological processes require highly pure water, which following the production cycle, is much purer than raw water. In this case, the application of a water circulation system is profitable because additional treatment and return to technological processes of used water is more efficient than treatment of tap water /2/. The use of water recirculation systems decreases fresh-water consumption and, consequently, water treatment costs. Recirculated water treatment becomes the main water-treatment process, thus reducing capital and operational expenses on this process as well as on sewerage systems. At chemical plants, the introduction of water recirculation systems secured more than 35% reduction in costs /1/.

Water recirculation systems are being increasingly used in the USSR. On the average for the country, the use of water from these systems in 1985 accounted for 72.5% and, excluding thermal power stations, more than 80% of total water consumption, the total capacity of recirculation systems amounted to about 250 km^3/year, i.e., about as much as the mean annual runoff of such a river as the Volga /6/.

One of particularly water-consuming industries is ferrous metallurgy (water consumption in this industry accounts for 15% of the country's total industrial water consumption). Large ferrous metallurgical works use up to 200-300 thousand m^3/hr each /10/.

Today, the country's average in the use of water recirculation systems in the above complex industries is about 90% /10/.

It is to be emphasized that some metallurgical works in the USSR are already operating water recirculation systems, i.e., disposal of waste water into water bodies has been completely eliminated there, while intake of fresh water is made solely to compensate for the losses in the recirculation systems. The economic benefit from elimination of waste water disposal by metallurgical works amounts already to 4 million rubles annually /8, 9/. In pulp-and-paper industry, which accounts for about 9% of total industrial water consumption, certain difficulties were encountered until recently in setting up recirculation systems, particularly in bleached pulp production. For all the pulp-and-paper industry, recirculated water consumption averages at present about 65%, however in cardboard mills it is fairly high - 90-95% /4/.

A good example of a production with a water recirculation system and with waste utilization in pulp-and-paper industry is the Suojärvi cardboard mill. Here, alongside the 10-time reduction in fresh-water consumption, the disposal of waste water into lake Suojärvi was completely stopped; this water formerly carried out organic substances, characterized with respect to BOD equalling 0.5 kg O_2/t of produce, with respect to COD equalling 0.6 kg O_2/t, nonvolatile water-soluble organic and mineral compounds - 16.2 kg, suspended substances - 0.8 kg per ton of produce. The excessive active sludge left over after the biological treatment of recirculated water is sent to pulp. In this case, the strength characteristics of cardboard, made from this pulp, are improved /4, 12/.

In the USSR, wasteless technologies and water circulation systems are being developed in pulp-and-paper industry on the basis of theoretical and practical works. The introduction of optimal water circulation systems is the first step in this direction. In nonleached sulphate pulp production the volume of water used in recirculation systems may be as high as 97%, in leached sulphate pulp production - 96%, in unleached sulphite pulp production - 85%, in newsprint production - 90%, in paper bag production - 98%, in printing paper production - 96%, and in cardboard production - 96% of the total water consumption volume /12/.

The fact that waste water and wastes from pulp-and-paper industry pose a serious danger to the environment makes it easy to understand how important are these developments.

There is occasionally observed in water recirculation systems a high concentration of salts which is due to partial evaporation of water during cooling, washing-out of salts from raw materials in production processes, as well to addition of coagulants and other chemicals. However, accumulation of salts in recirculated water does not increase indefinitely. In the majority of industrial plants, operating water recirculation systems, approximately after 10 water cycles dynamic equilibrium emerges between the inflow of salts and their removal from the water system, for instance, with adsorbing precipitates. The accumulation rate of salts and the advent of their stabilized content depend on a number of factors, such as the ionic composition of water, use of some chemicals and other factors.

At the Yenakievo and Makeevka iron and steel plants, the content of salts in fresh water amounts to 600 mg/l. On the attainment of dynamic equilibrium in the recirculation systems in these plants, the concentration of salts stabilizes at about 1500 mg/l. For hot rolling mills, the stabilization sets in at a higher salt content - 2000-3500 mg/l. However, the use of such mineralized water does not interfere with production operations. For gas cleaning on blast furnaces in metallurgical production an even higher concentration of salts in recirculated water can be tolerated: up to 30 and 40-50 g/l, respectively. Furthermore, contrary to the prevailing opinion, the rate of corrosion of production equipment with the use of highly-mineralized water in gas cleaning declines by 15% /10/.

Dressing plant operation with water recirculation systems has shown that even salt concentration as high as up to 6-7 g/l brings no harm to technological processes; what is more, it results in a higher efficiency of flotation extraction of useful constituents, as well as in a more effective precipitation of suspended particles.

Where the content of salts in recirculated water is limited, part of it (averaging 0.2–0.3 to 0.5% of total volume) is desalinated. For instance, at the Verkh-Isetsk metallurgical works as much as 0.5% of recirculated water is subjected to desalination; however, even the use for this purpose of a technologically obsolete evaporation plant, the cost of distillate produced at it being more than 1 ruble/m^3, does not reduce the total economic benefit from application of a recirculation system.

Work is in progress today for developing more effective and cheap methods of desalination. For instance, the cost of water desalination using the reverse-osmosis method with polyamide membranes is in the order of 30 kopecks/m^3.

An ecological effect is attained by using industrial water recirculation systems also from the viewpoint of formation of the so-called secondary pollution. This results from decreasing the release of pollutants because less rigorous requirements are imposed upon recirculated water treatment and because of treatment of a smaller volume of more concentrated water /2/.

Since many new industrial plants are being put into service and numerous old ones are being updated, to assess the water-use rate with introduction of water recirculation systems, being a multi-purpose water-conservation measure, there is a need to apply some technological parameters. First of all it is the water recirculation coefficient

$$K_{rec} = \frac{Q_{rec}}{Q_{was}} \cdot 100\% = \frac{Q_{rec}}{Q_{rec} + Q_{dis}} \cdot 100\% \qquad (1)$$

which is one of the basic technological parameters, determining what part of waste water (Q_{was}) of its total amount (Q_{rec} + + Q_{dis}) is used in water recirculation at the plant.

Earlier /11,13/, another formula was used as the water recirculation coefficient, which, in our opinion, more adequately describes the degree of rationality in using water at the plant and is known as the water-use rationality coefficient:

$$K_{rat} = \frac{Q_{rec} + Q_{irrev}}{Q_{rec} + Q_{dis} + K_{dil} \cdot Q_{dis} + Q_{los}} \qquad (2)$$

The lower water losses (Q_{los}) and the amount of waste water discharged into a water body (Q_{dis}), the more rational the use of water. Fresh-water consumption by the plant to meet utility requirements is not discussed here. It is essential that Q_{irrev} and Q_{los} are differentiated. Q_{irrev} represents useful consumption of water that is unavoidable in certain technologies. Q_{los} may be the total of evaporation and seepage from taling dumps and slime pits, being part of a water recirculation system, losses at cooling towers (evaporation and drop removal) and other losses. Water losses can and should be controlled since they may be occasionally substantial.

The use of formula (2) as the water recirculation coefficient cannot be correct because the Q_{irrev} and Q_{los} are not directly related to development of recirculation water supply.

Recently, use has often been made also of the water-use frequency coefficient /3/, which gives a significantly more clear-cut idea about the rate of water use in recirculation systems than K_{rec} does. It is

$$K_{fr} = \frac{Q_{rec} + Q_{dis}}{Q_{dis}} \qquad (3)$$

To take account of the extent of water recirculation attained at an industrial plant, amounts of irreversible water consumption and losses, the degree of treatment of waste water discharged into a water body, it is necessary that another parameter is introduced. It is the total volume of fresh water, used at an industrial plant for production purposes and for keeping the water body, where waste water is discharged, in a normal condition /7/:

$$Q_{fr.total} = Q_{fr} + Q_{dil} = Q_{irrev} + Q_{los} + Q_{los.rec} + $$
$$+ Q_{dis}(1 + K_{dil}) \qquad (4)$$

where Q_{dil} is the amount of pure water needed for diluting waste water; K_{dil} is the coeficient of frequency of dilution of waste water with pure water to bring it to the standard quality.

Since the purpose of waste water dilution is to achieve the biological adequacy of water in the water bodies, it should at least meet the fishery requirements. Therefore, it was formerly suggested /6/ that Q_{dil} should be called "ecological water pass" the value of which should be appreciably larger than that of conventional sanitary water pass.

As the volume of pure water, used for diluting waste water, is directly related not only to its quantity, but also its quality, it should be taken into account in the water-economic budget. Accordingly, certain researchers (the authors, V.K. Papisov, and L.P. Sidorin) assume the quantity of fresh water, used for diluting waste water, to be an indicator of the degree of waste-water pollution:

$$P_{was} = Q_{was} + Q_{dil} = Q_{dis} + K_{dil} \cdot Q_{dis} = Q_{dis}\,(1 + K_{dil}) \qquad (5)$$

To calculate the value of Q_{dil} from formulas (4) and (5) with allowance for background pollution, we can easily derive the dilution coefficient:

$$K_{dil} = \frac{C_i - C_{ni}}{C_{ni} - C_{fi}} \qquad (6)$$

where C_i is the concentration of the i-th pollutant in the waste water discharged by the plant, C_{ni} is the permissible concentration of the i-th pollutant in the water body, C_{fi} is the background concentration of the i-th pollutant in the water used for dilution.

For industrial plants with a stable technology, K_{dil} for waste water can be determined by the biotesting procedure, e.g., daphnium testing; earlier the authors proposed the K_{dil} values of a certain degree of reliability for relevant real situations and for highly rough calculations /5/.

The total water-use rationality coefficient is derived from formulas (2), (4), (5) in the following form:

$$K_{rat.\,total} = \frac{Q_{rec} + Q_{irrev}}{Q_{rec} + Q_{los} + Q_{dis}\,(1 + K_{dil})} \qquad (7)$$

In analysing economic problems, involved in setting up water recirculation systems (comparison of recirculation systems and alternatives), we may use the methodological procedure, suggested in /5,6/, all the calculations being made in costs per unit water volume.

Introduction of complete water recirculation systems at industrial plants, apart from achieving ecological goals, produces, as a rule, a high economic benefit not only for the national economy at large, but also for individual industrial plants introducing water recirculation systems.

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BIOMASS TURN-OVER TIME AND WATER QUALITY.

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The reneval of the biomass takes place in the continuous flow biological systems. In these systems, the biomass output occurs simultaneously with the processes of biomass synthesis and input. The biomass may disappear from the system as the result of grazing by organisms of higher trophic levels, by sinking or washout with water flows, and so on /Uhlmann, 1971/.

The biomass turn-over time is the important general characteristics of a continuous flow biological systems.

The analysis of the models of biological communities /Vavilin, 1986/ reveals that when the biomass turn-over times are short (heavy loading), the slowly growing organisms, specifically adapted to consume hardly oxidized substrates, are displased from the system. Under these conditions the hardly assimilating substrates pass through the biological community in transit, without decomposition. When the biomass turn-over time is long (small loading) then a wide variety of substrates is utilized, because the slowly growing organisms adapted to utilize hardly assimilating substrates also remain in the system.

The statements are illustrates below by the models of lake ecosystems.

An abstract mathematical model of the lake ecosystem was analyzed presuming the existence of several species of bacteria and a few fractions of organic matter.

The model included one species of algae numbered i=1, five species of bacteria (i=2-6); one species of zooplankton (i=7), five fraction of organic matter (i=8-12), and mineral phosphorus. It was also assumed that the i-th species of bacteria was specialized to assimilate the organic matter fraction under index i+6 and that zooplankton utilized indiscriminately all bacterial species.

Three layers were considered in the water basin: the epilimnion (2.5 m thick, j=1), the hypolimnion (7.4 m thick, j=2), the bottom (0.1 m thick, j=3). It was suggested that only in the epilimnion that the input and the output of the continuous water flow occur.

The dynamics of the biotic components of the ecosystem in layer j (B_{ij}) was described by differential equations

$$\dot{B}_{ij} = R_{ij} - T_{ij} - C_{ij} - L_{ij} + D_j(B_{oi} - B_{ij}) + Q_{ij} \qquad (1)$$

where R_{ij} is the growth rate of algae and the ration for heterotrophs; T_{ij} is the respiration; C_{ij} is death rate; L_{ij} is grazing (different from zero only for bacteria); $D_j(B_{oi} - B_{ij})$ is the input and output of the relevant component with water flows (for the hypolimnion and the bottom it is zero), where B_{oi} is the concentration of the component in the influent flow, Q_{ij} is the exchange with the neighbouring layers by sinking.

The value R_{ij} is presumed to be equal to

$$R_{ij} = a_i B_{ij} AL \qquad (2)$$

where a_i is the growth constant for algae and the food assimilation constant for heterotrophs; AL is the restriction constant due the deficiency of substrate. This value was calculated by different formulas according to the kind of organisms. They are:

1) for the algae in the epilimnion

$$AL = \frac{1}{z_1} \int_0^{z_1} \min(P_1/(P_1+K_P),\ I(z)/(I(z)+K_I))\, dz \qquad (3)$$

where P_1 is concentration of the mineral phosphorus; $I(z)$ is intensity of light at depth z; K_P and K_I are constants of half-saturation for phosphorus and light; z_1 is the depth of the epilimnion;

2) for the algae in the hypolimnion and on the bottom AL=0;

3) for bacteria

$$AL = \sum_{k=8}^{12} r_{ik} B_{kj})/((\sum_{k=8}^{12} r_{ik} B_{kj}) + K_{Bi}) \qquad (4)$$

where B_{kj} is the concentration of k-th organic substance in j-th layer, $r_{ik}=1$ if k-th organic substance consumed by i-th bacteria and $r_{ik}=0$, if not consumed, K_{Bi} is the half-saturation constant;

4) for zooplankton

$$AL = (\sum_{k=2}^{6} B_{kj})^2/((\sum_{k=2}^{6} B_{kj})^2 + K_{B7}^2) \qquad (5)$$

where $\sum B_{ij}$ is the sum of concentrations of all bacterial species, K_{B7}-the half-saturation constant.

Intensity of light in the epilimnion at depth z is described as

$$I(z)=I(0)\exp(-\varepsilon_0 z-\sum_{i=1}^{12}\varepsilon_i B_{i1} z) \qquad (6)$$

where I(z) is the intensity of light on the depth z (I(0) - on the surface), ε_0 and ε_i are the coefficients of the water extinction and of the biotic components extinction.

The following formula was used for respiration:

$$T_{ij}=u_i B_{ij}+(1-y_i)R_{ij} \qquad (7)$$

where u_i are the respiration constants, y_i is the yield coefficient, which for algae was equal to 1.

The death rate was represented by the formula

$$C_{ij}=c_i B_{ij} \qquad (8)$$

where c_i is the death rate constant, and bacteria grazing by the formula

$$L_{ij}=R_7 B_{ij}/\sum_{k=2}^{6} B_{kj} \qquad (9)$$

where R_7 is the zooplankton ration.

The following equation was considered valid for the organic water fractions:

$$\dot{B}_{ij}=(\sum_{k=1}^{7} C_{kj})H_i - L_{ij} + D_j(B_{oi}-B_{ij}) + Q_{ij} \qquad (10)$$

where $D_j(B_{oi}-B_{ij})$-is the input and output of the component with water flow, Q_{ij} is the exchange with the neighbouring layers, L_{ij} is the utilization of i-th fractions of organic substance by bacteria, described as

$$L_{ij}=(\sum_{k=2}^{6} R_{kj} r_{ki} B_{kj}/(\sum_{l=8}^{12} r_{kl} B_{lj})) \qquad (11)$$

$(\sum_{k=1}^{7} C_{kj})H_i$ is the formation of organic matter fractions in the process of decay of biotic components (H_i is the content of i-th fraction of organic matter in any biotic component).

The dynamics of phosphorus is described by equation

$$\dot{P}_j=\sum_{k=1}^{12} U_{kj}\gamma_k+D_j(P_o-P_j)+Q_{Pj} \qquad (12)$$

where U_{kj} is the rate of changes in the concentration of biotic components and organic matter as the result of biological processes, γ_k is phosphorus content in k-th component (every component have the same phosphorus content), $D_j(P_o-P_j)$ is phosphorus input ti the epilimnion and phosphorus output with the water flow, Q_{Pj} is exchange with the neighbouring layers.

It is obvious that value U_{kj} exactly corresponds to the amount of phosphorus which was transferred from the water phase to the biotic components in the unit of volume for a unit of time.

The value Q_{ij} (and also Q_{Pj}) was assumed equal to

$$(K_{df,i}(B_{i2}-B_{i1})-K_{sc,i}B_{i1})/z_1 \quad \text{-for upper layer,}$$

$$(K_{df,i}(B_{i1}+B_{i3}-2B_{i2})+K_{sc,i}(B_{i1}-B_{i2}))/z_2 \quad \text{-for intermediate layer,}$$

$$(K_{df,i}(B_{i2}-B_{i3})+K_{sc,i}B_{i3})/z_3 \quad \text{-for lower layer.}$$

$K_{df,i}$ is the constant of diffusion and $K_{sc,i}$-the constant of sinking.

Several constants were used in calculations(see Table 1).

Three variants of the dynamics of the ecosystem components during a 150 days period were calculated by the computer (Tables 2-3). In the first calculation set, imitating a mesotrophic lake the mineral phosphorus concentration in the inflow water was selected in such a way as to obtain 40 mg/m^3 of the total phosphorus concentration, in the second calculation set (eutrophic lake) the total concentration reached 400 mg/m^3, and in the third calculation set (hypertrophic lake) it was 4000 mg/m^3.

When analysing the obtained results, we should first of all note that the present paper is not aimed at achieving quantitative coincidence of calculations with the experimental data. Its foremost task is to demonstrate that the eutrophication of the lake may cause simplification of the specific structure of the bacterial community by displacing the slow growing species adapted to consume hardly oxidized organic matter. This process is the result of zooplankton grazing on bacteria thus causing forced removal of bacterial biomass.

Fig. 2 shows the distribution of bacteria in the lake after a 150 days period. It is evident that the eutrophication of the lake results in the reduction of the bacterial diversity in the epilimnion and hypolimnion, but in the hypolimnion remain the bacteria consuming more hardly oxidizable organic matter. This happens because easily oxidizable organic matter is mostly decomposed in epilimnion. In all three cases in the bottom layer the fast growing species dominated, because organic matter accumulated at the bottom.

The structure of bacterial communities could be characterized by biomass turn-over rate, determined as

$$V_{to} = \sum_{i=2}^{6} R_{ij} \Big/ \sum_{i=2}^{6} B_{ij} \qquad (13)$$

and by biomass turn-over time $T_{to} = 1/V_{to}$.

The turn-over time decreased with eutrophication, it decreased in seria "hypolimnion-epilimnion-bottom".

Let us compare the data on the content of different organic matter fraction. If in the first variant (mesotrophic lake) the concentrations of all the organic matter fractions in the epilimnion are fairly low, then in the second and third variants (eutrophic and hypertrophic lakes) the concentrations of the third, fourth and fifth organic matter fractions are 4-5 times higher than the concentration of the easily oxidized first fraction. This is attributed to the absence of bacteria assimilating hardly oxidizable fractions in the epilimnion; consequently these fractions pass through the lake"in transit".

In order to describe the input-output relations, the following formula is often used (Vollenweider, 1975):

$$P_{outp}/P_{inp} = 1/(1+\sqrt{\tau}) \qquad (14)$$

Its modification is

$$P_{outp}/P_{inp} = a/(1+b\sqrt{\tau}) \qquad (15)$$

where P_{inp} is the total amount of phosphorus in inflowing water, P_{outp} is the total amount of phosphorus in outflowing water (in our case-in epilimnion), τ -the time of water exchange in years, a and b are constants.

As evident from Fig. 3, in the modelled system at P_{inp} = 400 mg/m^3 for not too long turn over times the dependence of P_{inp}/P_{out} on $\sqrt{\tau}$ is linear, which is in conformity with the generalized Vollenweider formula (15). In our case a=1.2 and b= 1.6.

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Table 1. Constants used in the lake ecosystem model*

	Biota						
	1	2	3	4	5	6	7
a_i, day^{-1}	2.0	2.5	2.0	1.5	1.0	0.5	0.5
c_i, day^{-1}	0.2	0.25	0.20	0.15	0.10	0.05	0.05
u_i, day^{-1}	0.2	0.25	0.20	0.15	0.10	0.05	0.05
y_i		0.5	0.5	0.5	0.5	0.5	0.5
K_{Bi}, units/m^3		0.2	0.2	0.2	0.2	0.2	0.2
K_P, mg/m^3	10						
K_I, ill, units	0.01						
γ_i, mg/units	20						
ε_i, m^2/units	0.2	0.1	0.1	0.1	0.1	0.1	0.1
K_{wc}, m/day	0.0	0.0	0.0	0.0	0.0	0.0	0.0
K_{df}, m/day	0.01	0.01	0.01	0.01	0.01	0.01	0.01
$B_i(t=0)$ units/m^3	0.1	0.1	0.1	0.1	0.1	0.1	0.1
B_{oi}, units/m^3	0.001	0.001	0.001	0.001	0.001	0.001	0.001

	Organic Matter				
	8	9	10	11	12
H_i	0.2	0.2	0.2	0.2	0.2
K_{wc}, m/day	0.2	0.2	0.2	0.2	0.2
K_{df}, m/day	0.01	0.01	0.01	0.01	0.01
γ_i, mg/units	10	10	10	10	10
ε_i, m^2/units	0.1	0.1	0.1	0.1	0.1
$B_{ij}(t=0)$ units/m^3	0.1	0.1	0.10	0.1	0.1
B_{io}, units/m^3	0.1	0.1	0.1	0.1	0.1

General constants

D_i = 0.05 day^{-1}

I_o = 30.0 ill.unit

ε_o = 0.5 m^{-1}

---

* Biomass and organic matter concentration are determined in conventional units

Table 2. Variant 1 (concentration of different variables) Mesotrophic Lake, 40 mg/m^3 of phosphorus in the influent water

	epilimnion	hypolimnion	bottom
algae	1.22	0.0	0.0
bacteria:			
1 species	0.029	0.003	0.054
2 species	0.034	0.004	0.046
3 species	0.032	0.007	0.044
4 species	0.031	0.015	0.024
5 species	0.004	0.050	0.016
bacterial biomass	0.13	0.081	0.185
turn-over rate	0.64	0.20	0.97
zooplankton	0.113	0.020	0.237
total biomass	1.46	0.109	0.422
organic matter			
1 fraction	0.095	0.050	0.164
2 fraction	0.11	0.051	0.233
3 fraction	0.136	0.054	0.368
4 fraction	0.205	0.060	0.881
5 fraction	0.423	0.074	1.38
total organic matter	0.97	0.29	3.03
total biomass +			
organic matter	2.43	0.39	3.45
mineral phosphorus	3	28	10
organic phosphorus	39	5	39
total phosphorus	42	33	49

Table 3. (concentrations of the different variables) Hypereutrophic lake; 4000 mg/m^3 of phosphorus in the influent water

	epilimnion	hypolimnion	bottom
algae	39	0.145	0.032
bacteria:			
1 species	0.112	0.060	0.096
2 species	0.002	0.035	0.009
3 species	0.0	0.00	0.00
4 species	0.0	0.00	0.00
5 species	0.0	0.00	0.00
bacterial biomass	0.114	0.096	0.106
turn-over rate	2.42	2.02	2.40
zooplankton	0.87	0.81	0.93
total biomass	40	1.05	1.06
organic matter			
1 fraction	7.05	0.95	15.7
2 fraction	12.3	19.5	350
3 fraction	12.8	35.5	635
4 fraction	12.9	35.9	643
5 fraction	13.0	40.4	730
total organic matter	58.1	132	2370
total biomass + organic matter	98.0	133	2380
mineral phosphorus	2230	2860	2660
organic phosphorus	1380	1340	23740
total phosphorus	3610	4200	26400

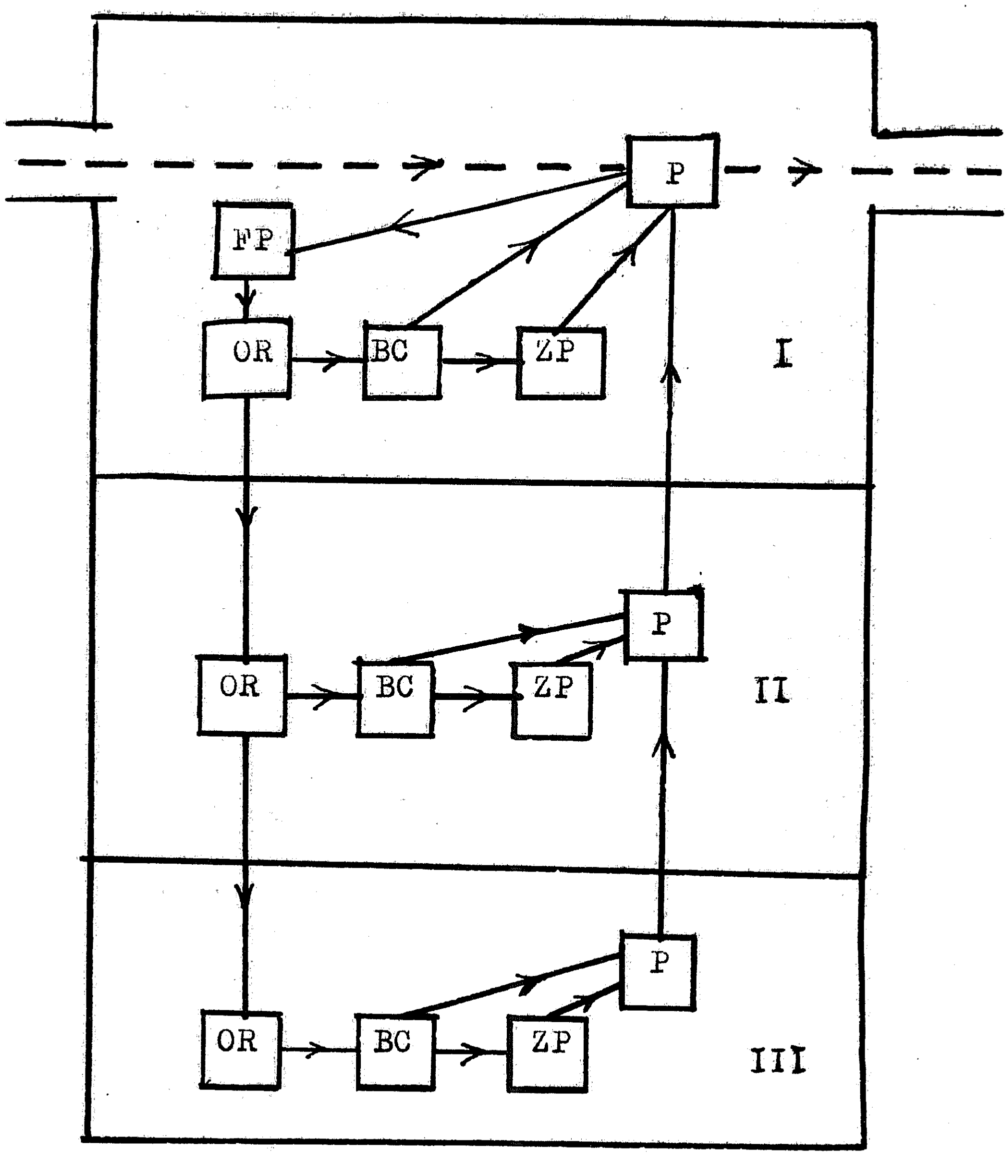

Fig. 1.

The block-scheme of the lake ecosystem model; I-epilimnion, II-hypolimnion, III-bottom.
FP-algae, BC-bacteria, ZP-zooplankton (microzoobenthos at the bottom), OR-organic matter, P-mineral phosphorus.

The scheme does not show the production of organic matter by decay of bacteria and zooplankton and the removal of phosphorus as the result of algae and bacterial respiration.

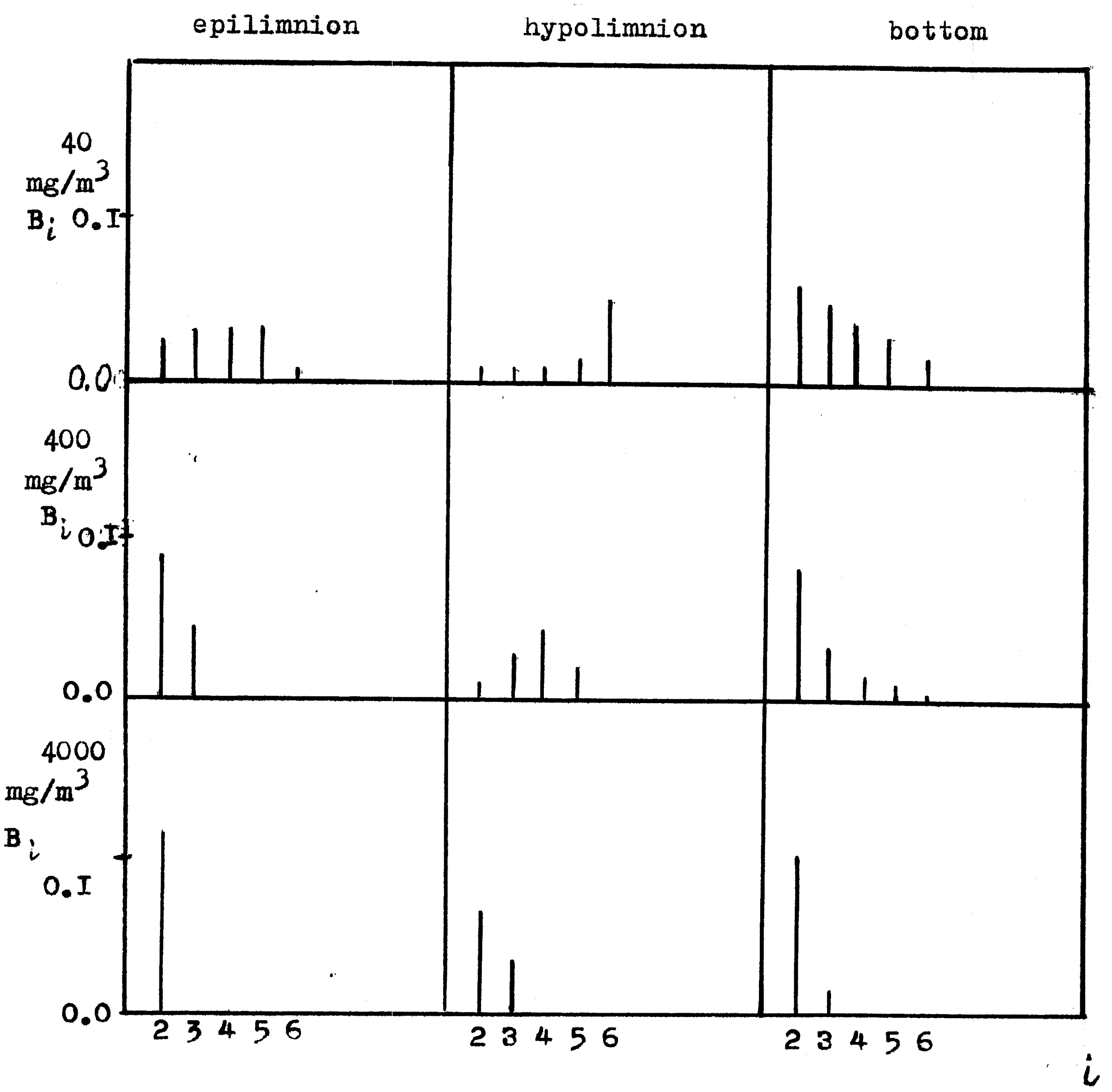

Fig. 2.

Bacterial Biomass composition after 150-days period (the species are numbered from left to right). The first column represent epilimnion, the second column-hypolimnion, the third column-bottom. The first line shows variant 1, the second line-variant 2, the third line-variant 3.

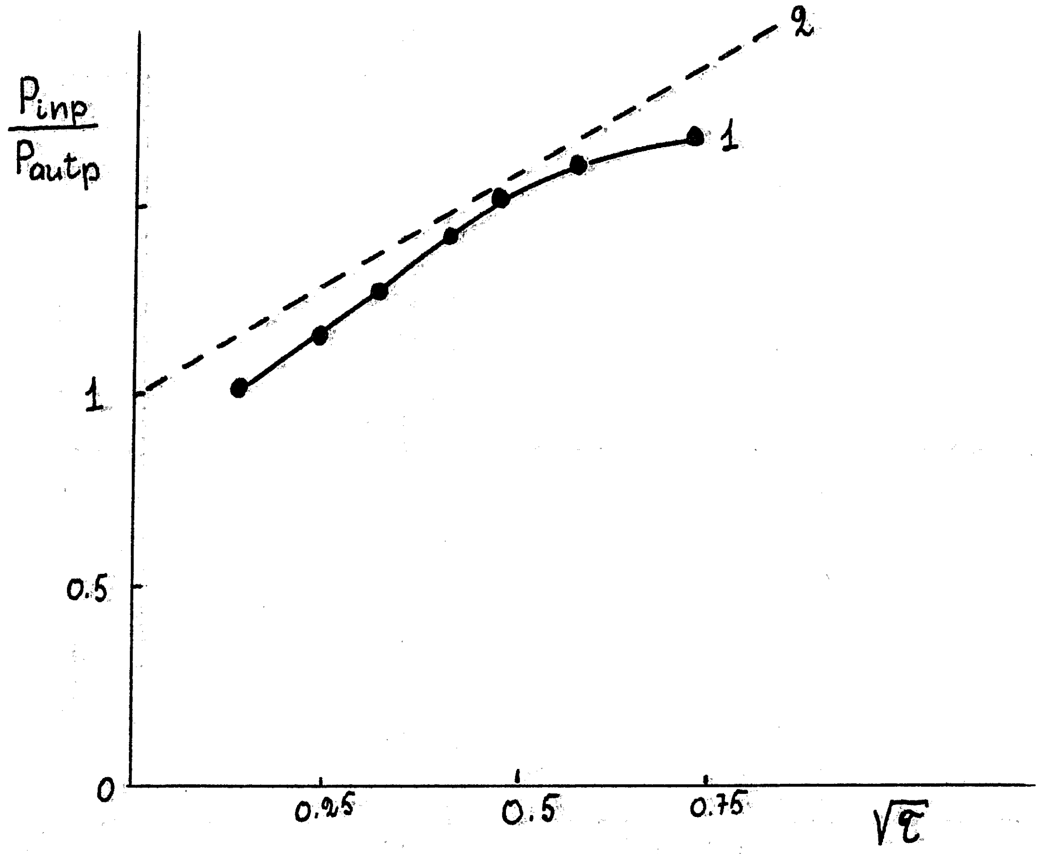

Fig. 3.

The dependence P_{inp}/P_{out} on $\sqrt{\tau}$, P_{inp} =400 mkg/l.

1-calculated dependence

2-Vollenweider dependence

SEAWATER INTRUSION INTO COASTAL FRESH-WATER AQUIFERS AND ITS EFFECT ON GROUND WATER QUALITY

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In case of submarine discharge of ground water within interface of fresh ground water and saline seawater a transition zone arises with highly variable water salinity ranging from fresh to saline seawater. Location and size of this zone are dictated by seawater density, the rate of subaqueous discharge of ground water and other factors. Construction of water intake structures in littoral regions and intensified pumpage of ground water can lead to seawater intrusion into fresh-water aquifers and can give rise to combatting problems in respect to the pollution of ground water sources.

For the purpose of effective management of aquifers in littoral regions and prevention of their pollution a flexible mathematical model is indispensable the describes combined movement of fresh and saline water in an aquifer. Mathematical modelling of seawater intrusion can be subdivided into two groups: in one it is a assumed that there is a distinct boundary between fresh and saline water, in the other - that two liquids are mixing. Both groups of the tasks yield information specifying the relevant method to be applied in each particular case.

1. Solution of the task concerning identification of interface for the model of movement of two non-mixing liquids can be done by several different techniques. The ratio

$$H = \frac{\rho_f h}{\rho_m - \rho_f} \qquad (1.1)$$

for unconfined aquifer was obtained from the condition of hydrostatic equilibrium by the method known as Gbyben-Herzberg lens, where h is elevation of fresh water above sea level,

H is depth of interface below sea level,
ρ_f, ρ_m are densities of fresh and saline water, respectively.

At ρ_m = 1.025 g/cm^3, ρ_f = 1.00 g/cm^3 we obtain from (1.1) H = 40 h. But since fresh ground water flows into the sea at a certain finite velocity the location and shape of fresh and saline water interface should be governed by conditions of their dynamic interaction. However, for quasihydrostatic condition at a considerable distance from sea shore where fresh water flow is almost horizontal, the ratio (1.1) is to an adequate degree applicable.

The method that takes infiltration into account while identifying the shape of interface in coastal aquifers is based on Dupuit approximation about horizontal flow in aquifers. For steady-state flows in unconfined aquifer the Dupuit formula as based on the Ghyben-Herzberg ratio will take the form

$$Q + Rx = - kh\ (1 + \theta)\, \partial h/\partial x, \qquad (1.2)$$

where R is infiltration, $\theta = (\rho_m - \rho_f)/\rho_f$

Boundry conditions

$x = 0,\ h = h_o;\ x = L,\ h = 0.$

By integrating the equation (1.2) one can obtain the equation for determining the boundary of interface

$$h^2 = h^2_o - \frac{2Qx + Rx^2}{k\ (1 + \theta)} \qquad (1.3)$$

$h^2_o = (2QL + RL^2)/k(1 + \theta)$

The obtained ratios reveal the relationship between depth of saline water intrusion L, Piezometric head h_o of fresh water over interface surface and discharge Q of fresh water flow.

Application of the Dupuit approximation is justified only in the case when vertical velocities can be discharded which is not valid in the vicinity of sea shore.

Improved analytical solutions in respect to homogeneous confined aquifers are presented in the works [2 - 4] using methods of hodograph and double series of Fourier. The findings of experimental investigations show the non-mixing liquids are separated by a macroscopically distinct surface of interface, and both non-mixing liquids occupy separate areas that do not cross each other. The shape of interface surface and its location may continuously vaaery in space. Both liquids are distinguished according to their physical properties: density, viscosity, etc.

If the movement of two non-mixing liquids in porous medium occurs in such a way that one liquid is tagnant, while the other is in the steady-state plane flow, then analytiucal solution can be obtained using the method of hodograph. Although in this case interface of two liquids is not known in advance, but the boundary condition is known, consequently, location of interface becomes known in the plane of hodograph.

Let us consider vertical the section of a fresh-wter aquifer that in perpendicular to sea shore where fresh water of γ_f specific weighty moves above stagnant saline liquid of γ_m specific weight (Fig. 1).

Let us introduce complex plane $z = x + iy$ and complex potential of fresh water flow $w_1 = \varphi + i\Psi$, where Ψ is function of current, 4 is potential of specific discharge of fresh water flow

$$\gamma = k_f \left(y + \frac{p}{\gamma_f}\right), \quad k_f = \frac{k\gamma f}{\mu f} \qquad (1.4)$$

k_f is constant hydraulic copnductibity of porous medium with respect to fresh water,

μf is viscosity of fresh water.

Boundary conditions for complex potential w_1 have the following form:

on AF line which is the line of current,

$$\Psi = 0, \quad y = 0; \qquad (1.5)$$

on FE line which is also the line of current,

$$\Psi = Q, \quad y = -L, \qquad (1.6)$$

where Q is fresh water discharge per unit of aquifer thickness.

AB line is the boundary between the percolation zone and water body, therefore, on this line it can be assumed for the case of small angles α that

$$\varphi = 0, \quad y = -x \operatorname{tg} \alpha\pi. \qquad (1.7)$$

BE line being the line of current simultaneously separates moving fresh water from stagnant saline. Accordingly, along this line [5] we have

$$\Psi = Q. \quad \varphi = -k'y, \qquad (1.8)$$

where

$$k' = k_f (\gamma_m - \gamma_f)/\gamma_f$$

On w_1 plane the semiband AFEB will correspond to AFEB flow area in realt physical plane z.

In plane w_1 point E has coordinates $\Psi_E = Q$, $\varphi_E = k'(L + Y_E)$.

Let's introduce complex velocity of flow $w = u + iV$, (1.9)

where components of liquid flow velocity are determined by the formulas:

$$u = -\frac{\partial\varphi}{\partial x}, \quad V = -\frac{\partial\varphi}{\partial y} \qquad (1.10)$$

Complex plane w is usually called the plane of current hodograph. Using results of the work [5] we can show fresh water flow on the plane of w hodograph.

In plane w the BE boundary is part of circumference with k radius and centre having coordinates $u = 0$, $V = k'/2$. Velocity at F point is equal to $u_F = Q/L$.

Later on, complex transition velocity $w = u - iV$ will be required, therefore we show the flow area in w plane onto w plane which is a mirror image in relation to u axis.

By applying transformation $w' = \frac{k'}{w}$ we transform curvilinear area into rectilinear one.

Flow areas on w_1 and w' planes are conformally reflected into auxiliary complex plane ξ (onto its upper part).

Using the Kristoffel-Schwartz formula [5] we obtain parametric connection of complex potential plane with plane of hodograph

$$w'(\xi) = \frac{1}{\pi}\int_1^{\xi}\left(\frac{\xi + 1}{\xi - 1}\right)^{\alpha}\frac{d\xi}{\sqrt{\xi^2 - 1}}, \qquad (1.11)$$

$$w_1(\xi) = -\frac{Q\sqrt{a^2 - 1}}{\pi}\int_1^{\xi}\frac{d\xi}{(\xi - a)\sqrt{\xi^2 - 1}} \qquad (1.12)$$

Since

$$\frac{dw_1}{dz} = -w, \qquad (1.13)$$

stemming from it we obtain conformal reflection of auxiliary plane ξ on the flow area in physical plane z.

$$\frac{dz}{d\xi} = \frac{Q\sqrt{a^2 - 1}}{\pi k'}\,\frac{w'(\xi)}{(\xi - a)\sqrt{\xi^2 - 1}} \qquad (1.14)$$

In order to determine the unknown parameter of conformal reflection "a", we make use of F points correspondence on w' plane and on the auxiliary plane ξ

$$\frac{1}{B1} = \frac{\pi k' L}{Q} = \int_1^{a}\left(\frac{\xi + 1}{\xi - 1}\right)^{\alpha}\frac{d\xi}{\sqrt{\xi^2 - 1}} \qquad (1.15)$$

By integrating the ratio (1.14) in the range from $\xi = -1$ to $\xi = +1$, and introducing dimensionless variables

$$\overline{x} = \frac{x}{L},\ \overline{y} = \frac{y}{L},\ \overline{x}_B = \frac{x_B}{L},\ \overline{y}_B = \frac{y_B}{L},$$

we find coordinate of B point on tha plane z:

$$\overline{x}_B + i\overline{y}_B = \frac{B_1\sqrt{a^2 - 1}}{\pi}\,e^{-i\alpha\pi}\int_{-1}^{1}\frac{f(\xi)\,d\xi}{(a - \xi)\sqrt{1 - \xi^2}} \qquad (1.16)$$

where

$$f\,(\xi) = \int_{\xi}^{1} \left(\frac{1 + t}{1 - t}\right)^{\alpha} \frac{dt}{\sqrt{1 - t^2}}$$

The equation for determining "a" parameter (1.15) is transformed into

$$\frac{1}{B_1} = \int_{1}^{a} \left(\frac{\xi + 1}{\xi - 1}\right)^{\alpha} \frac{d\xi}{\sqrt{\xi^2 - 1}} \qquad (1.17)$$

whereas equations of the interface line - into:

$$\bar{y} = \bar{y}_B - B_1 \cdot \ln\left[\frac{ap + 1}{p + a} + \left(\left(\frac{ap + 1}{p + a}\right)^2 - 1\right)^{1/2}\right], \qquad (1.18)$$

$$x = \bar{x}_B - B_1\sqrt{a^2 - 1} \int_{a}^{chp} \left[tg\alpha\pi + \int_{0}^{u} \left(\frac{ch\chi - 1}{ch\chi + 1}\right)^{\alpha} d\chi\right] \frac{du}{chu + a}$$

In Fig. 2 some results of calculations are presented concerning the shape of stagnant saline wedge using formulas (1.18). As shown the size of intrusion area and depth of seawater penetration into confinced coastal fresh-water aquifer is apparently dependent upon sea bottom gradient. Thus, at one and the same value of parameter B1 = 0.3 the depth of saline water penetration decreases more than twice with increasing of sea bottom gradient from 0° to 15°.

2. However, as is known, fresh and saline water are mixing liquids, and a more or less extended dispersion zone, as a rule, lies between them, the size of which as dictated by hydrogeological conditions ranges from several metres to hundreds of metres. Variation of water salinity in the transitional dispersion zone, especially, in the case of its comparatively large extent, is to be taken into consideration in designing water intake structures in the zone of coastal aquifers [6].

Let us consider the section of a confined aquifer two-dimensional in vertical plane and perpendicular to sea shore (Fig. 1). Basic equations of the process of saline water intrusion into such aquifer are as follows: Dercy law, continuity equation for liquid, equation of dissolved salt transfer, equation of liquid state.

Seawater movement in this case takes the form of a saline water wedge penetrating an aquifer in its lower part. At a point where sea-water encounters fresh water counterflow mixing of them occurs. Resultant mixture, being less heavy than seawater, rises upwards and drifts towards the sea under the influence of fresh water presusure gradient. Accordingly, a convective cell of brackish water is created in the aquifer. As soon as the amount of dissolved salt entering throught the lower part of sea boundary of the aquifer is balanced by the amount of salt carried away through the upper part of the boundary, stationary dynamic equilibrium is attained at the stationary boundary conditions.

While selecting x, y axes of coordinate system (Fig. 3) in the direction of main axes of anisotrophy, and bearing in mind that $\theta = \frac{\gamma_s - \gamma_f}{\gamma_f} \ll 1$, the Darcy equation and those of continuity for the stationary case are written as follows

$$u = -\frac{Kx}{\mu}\frac{\partial P}{\partial x},$$

$$w = -\frac{Ky}{\mu}\left(\frac{\partial P}{\partial y} + \rho g\right) \qquad (2.1)$$

$$\frac{\partial u}{\partial x} + \frac{\partial w}{\partial y} = 0$$

where u, w are seepage velocities towards x, y axes, respectively, γ_s, γ_f are specific weights of saline and fresh water.

Introducing Ψ current function by the ratio

$$u = \frac{\partial \Psi}{\partial y}, \quad w = -\frac{\partial \Psi}{\partial x} \qquad (2.2)$$

and assuming permeability and dispersion coefficients to be constant, we transform equation system (2.1) and diffusion equation to the following form

$$\frac{\partial^2 \Psi}{\partial y^2} + \frac{Kx}{Ky}\frac{\partial^2 \Psi}{\partial x^2} = -\frac{\theta Kx\gamma_f}{\mu}\frac{\partial c}{\partial x}, \qquad (2.3)$$

$$\frac{\partial \Psi}{\partial y}\frac{\partial c}{\partial x} - \frac{\partial \Psi}{\partial x}\frac{\partial c}{\partial y} = Dx\frac{\partial^2 c}{\partial x^2} + Dy\frac{\partial^2 c}{\partial y^2}$$

By introducing dimensionless variables

$$\Psi = \bar{\Psi}Q, \quad x = \bar{x}l, \quad y = \bar{y}L \qquad (2.4)$$

where l is characteristic length of intrusion zone, L is thickness of the aquifer.

Let's substitute (2.4) into (2.3) and equate coefficients at first and third members of the first equation in (2.3) (balance of viscosity and flowability forces). Then we obtain

$$C = L\ Ra/Pe \qquad (2.5)$$

where

$Ra = K_x L \theta \gamma f$ is Rayleigh number,

$Pe = Q/D_y$ is Peclet number.

Omitting vinculum over variables, we rewrite the system (2.3) in the form

$$\frac{\partial^2 \Psi}{\partial y^2} + b^2 \frac{K_x}{K_y} \frac{\partial^2 \Psi}{\partial x^2} = - \frac{\partial c}{\partial x}, \quad b = \frac{Pe}{Ra}$$

$$\frac{\partial^2 c}{\partial y^2} = b\, Pe \left(\frac{\partial \Psi}{\partial y} \frac{\partial c}{\partial x} - \frac{\partial \Psi}{\partial x} \frac{\partial c}{\partial y}\right) - \frac{D_x}{D_y} b^2 \frac{\partial^2 c}{\partial x^2}. \qquad (2.6)$$

According to the data presented in the work [7] for aquifers composed of limestones and coarse.grained sand, "b" parameter is equal to 0.04. In this case, in equations (2.6) members with b^2 coefficient can be disregarded, provided $D_x \approx D_y$, $K_x \approx K_y$. The system of equations will be presented in a simplified form

$$\frac{\partial^2 \Psi}{\partial y^2} = - \frac{\partial c}{\partial x}, \qquad (2.7)$$

$$\frac{\partial^2 c}{\partial y^2} = b\, Pe \left(\frac{\partial \Psi}{\partial y} \frac{\partial c}{\partial x} - \frac{\partial \Psi}{\partial x} \frac{\partial c}{\partial y}\right)$$

with boundary sonditions

$$\Psi = 0, \ \frac{\partial c}{\partial y} = 0, \ y = 0 \qquad (2.8)$$

$$\Psi = 1, \ \frac{\partial c}{\partial y} = 0, \ y = 1$$

$\lim c = 0$ at $x \to - \infty$.

Approximated analytical solution of (2.7) - (2.8) was obtained in the work [8] in the form of a section of trigonometrical series for c(x, y)

$$c(x, y) = \sum_{k=0}^{n} c_k f_k(x) \cos K\pi y \qquad (2.9)$$

where $f_k(x) = \exp [(1 + k)\ mx]$; m, c_k are unknown constants. Function (3.9) satisfies all the boundary conditions on c(x, y).

In Fig. 3 the lines of equal salt concentrations within 0.1 - 0.9 are presented as well as the streamlines obtained in the work [8] at different values of n. With n increaseing the lines of equal salt concentration isochlores) somewhat shift towards fresh water, while trajectories of liquid particles rise to the top of the aquifer in the seaward end of the aquifer. The presence of return streamlines in the left lower angle - Fig. 3

indicates the circulation of seawater. In Fig. 3 isochlore c = 0.5 is presented that was obtained by Cooper and Pinder [9] using the method of characterization after attaining stationary state. Isochlore of Cooper and Pinder c = 0.5 in final points practically coincides with the given solution but its curvature is by far less.

Methods of analytical study of saline water encroachment can be used for qualitative evaluation of varying properties of coastal fresh-water aquifers at arbitrary values of key parameters.

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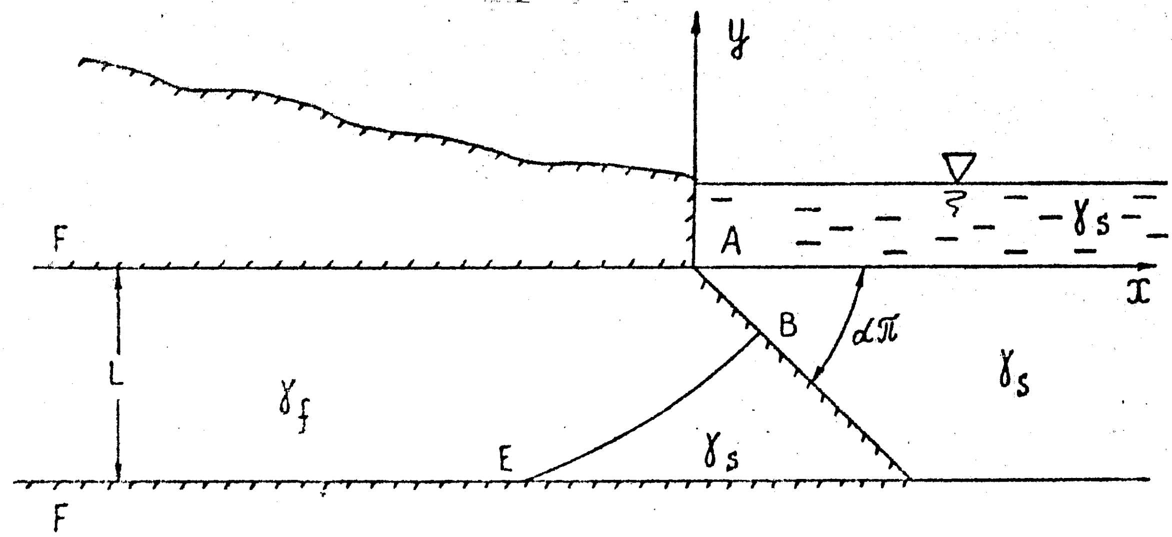

Fig.1. Schematic diagram of saline water intrusion.

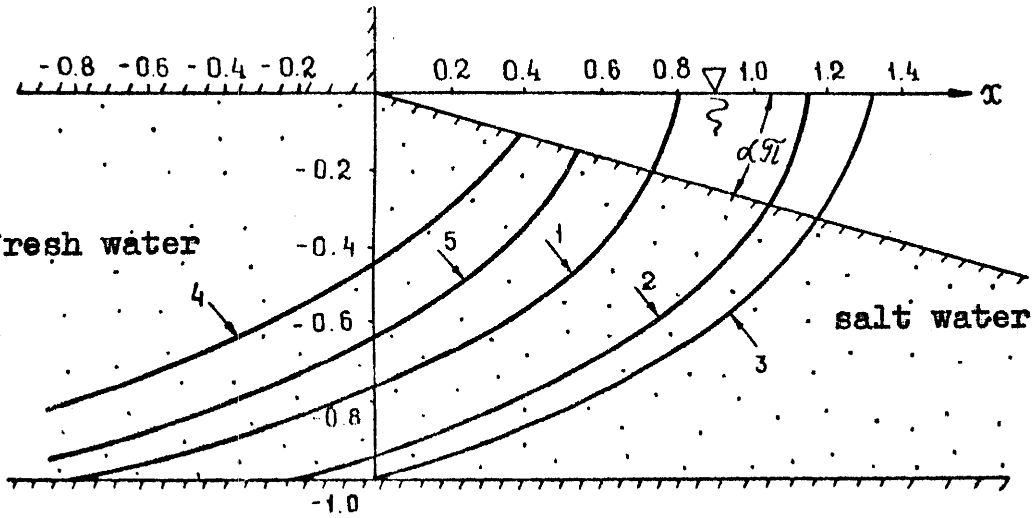

Fig.2. Fresh and saline water front: 1-3: α =0; 4-5: α =0.09. 1. B1=0.5; 2. B1=1.0; 3. B1=1.5; 4. B1=0.2; 5. B1=0.3.

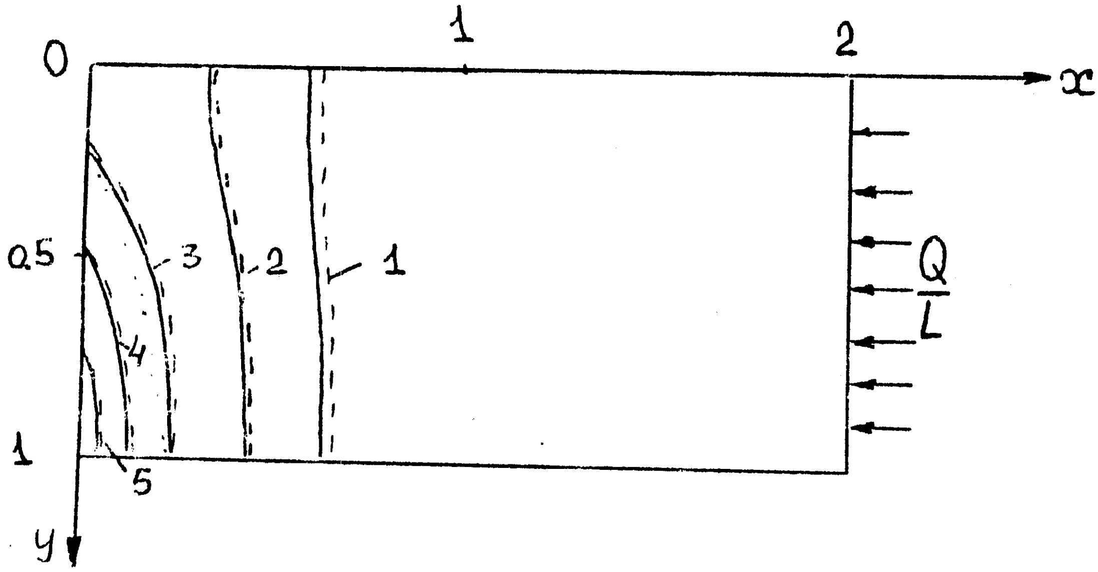

Fig.3. Distribution of salt concentration in unconfined coastal aquifer: 1 - C=0.1; 2 - C=0.3; 4 - C=0.7; 5 - C=0.9. ——— - n=2; - - - - n=4; ..... - C=0.5 according to[9].

SEA AND RIVER WATER MIXING IN ESTUARIES

M. G. Khublaryan and A. P. Frolov

The mixing of fresh river and salt sea waters in estuaries and river mouths has become a major subject of oceanological, hydrological, and ecologo-economic research because the intensive water intake from river drainage areas disturbs the natural equilibrium of water masses near the sea coasts.

Estuaries perform several vital functions. In particular, many water organisms stay there during their entire life cycles; it is the breeding ground for the juveniles of many fish species.

Hydrophysically, the most salient feature of an estuary is consistent increase in water salinity and density from the river end to the sea end of the estuary. The vertical salinity distribution and therespective mode of circulation flux is the main criterion in the hydrographic classification of valley type estuaries into wellmixed, partially mixed, and strongly stratified. In well-mixed estuaries the salinity does not practically vary depthwise; the fresh water is moved seawards and the sea water, landwards, by horizontal turbulent diffusion. At any depth the overall water flux is sea-bound.

1. Salt transfer in a well-mixedvalley type estuary is often described by one-dimensional mathematical models of convective diffusion which may be obtained by integrating three-dimensional nonstationary equations describing turbulent diffusion of solutes along the variable cross-section of the estuary.

The one-dimensional equation of convective diffusion has the form

$$\frac{\partial c}{\partial t} + V\frac{\partial c}{\partial s} = D\frac{\partial^2 c}{\partial s^2} + \frac{F'(s)}{F(s)} D\frac{\partial c}{\partial s} + q(s) \qquad (1.1)$$

where c is the additive concentration; t is the longitudinal (axial) coordinate; V is the fluid velocity; D is the constant factor of turbulent diffusion; q is the intensity of solute sources along the path; and F(s) is the crosssectional area or the estuary.

If the cross-sectional area varies expontentially

$$F(s) = F_o(S/S_o)^2 \qquad (1.2)$$

equation (1.1) can be rearranged into

$$\frac{\partial c}{\partial t} + V\frac{(\partial c)}{\partial s} = D\frac{\partial^2 c}{\partial s^2} + \frac{nD}{s}\frac{\partial c}{\partial s} + q \qquad (1.3)$$

where F_0 is the cross-sectional area with $S = S_0$ and n is the exponent.

An equation similar to (1.3) has been used in Ref. [1] to model the zone of pollution by nonconservative matter with a negligible advective transfer. In estuaries where the inflow of fresh water is noticeable this transfer cannot be ignored otherwise large errors may occur.

Let us take up several examples of solving equation (1.3) analytically. When there are no sources in a channel whose cross-sectional area is constant (n = 0) we have from equation (1.3)

$$\frac{\partial c}{\partial t} + V\frac{\partial c}{\partial s} = D\frac{\partial^2 c}{\partial s^2} \tag{1.4}$$

By introducing dimensionless parameters η and τ

$$s = \eta L,\ t = L^2\tau/D \tag{1.5}$$

where L is the characteristic linear scale, equation (1.4)

rearranges into

$$\frac{\partial c}{\partial \tau} + \beta\frac{\partial c}{\partial \eta} = \frac{\partial^2 c}{\partial \eta^2},\ \beta = \frac{VL}{D} \tag{1.6}$$

With boundary conditions

$$\tau = 0,\ c = 1;\ \eta = 0,\ c = 1;\ \eta = \infty,\ c = 0 \tag{1.7}$$

equation (1.6) is represented as

$$c(\eta,\ \tau) = \tfrac{1}{2}\left[\mathrm{erfc}\left(\frac{\eta + \beta\tau}{2\sqrt{\tau}}\right) + e^{\eta\beta}\ \mathrm{erfc}\left(\frac{\eta + \beta\tau}{2\sqrt{\tau}}\right)\right] \tag{1.8}$$

where

$$\mathrm{erfc}u = 1 - \mathrm{erf}u;\ \mathrm{erf}u = \frac{2}{\sqrt{\pi}}\int_0^4 e^{-\xi^2}\,d\xi$$

Equation (1.8) is often applied to heat conductivity and convective-diffusional transfer in porous media [2].

Let us use equation (1.4) to model convective-diffusional transfer of fresh river water in a mass of sea water with an allowance for evaporation. Introducing the dimnsionless variables η and τ of equation (1.5) the boundary-value problem

$$\frac{\partial c}{\partial \tau} + \beta\frac{\partial c}{\partial \eta} = \frac{\partial^2 c}{\partial \eta} - \alpha,\ \alpha = \frac{qL^2}{D}, \tag{1.9}$$

$$\tau = 0,\ c = 0;\ \eta = 0,\ c = 1;\ \eta = 1,\ c = 0$$

is solved as a trigonometric series

$$c = \bar{c} + \sum_{k=1}^{\infty} B_{1k} \exp\left(\frac{\beta\eta}{2} - \lambda^2_k \tau\right) \sin k\pi\eta \qquad (1.10)$$

where

$$\lambda^2_k = k^2\pi^2 + \frac{\beta 2}{4}, \quad \bar{c} = 1 - \frac{\alpha 2}{\beta} - \frac{\left(1 - \frac{\alpha}{\beta}\right)\left(1 - \exp\beta\eta\right)}{1 - \exp\beta},$$

$$B_{1k} = \frac{8\pi k}{4\pi^2 k^2 + \beta^2}\left[\frac{\alpha}{\beta}(-1)^{k+1}\pi \exp\left(\frac{\beta}{-2}\right) - \frac{4\pi\beta}{4\pi^2 k^2 + \beta^2}\left(1 + \right.\right.$$

$$\left.\left. + (-1)^{k+1}\exp(-\beta/2)\right] + \left(\frac{1 - \frac{\alpha}{\beta}}{1 - \exp\beta} - 1\right)\left[1 + (-1)^{k+1}\exp\right.$$

$$\left.(-\beta/2)\right] - \frac{1 - \frac{\alpha}{\beta}}{1 - \exp\beta}\left[1 + (-1)^{k+1}\exp\beta/2\right]$$

and q is the constant evaporation rate minus precipitation ($q > 0$).

If, on the left-hand side, $\eta = 0$, the boundary condition of the third kind is specified

$$\frac{\partial c}{\partial\eta} + \beta(1 - c) = 0 \qquad (1.11)$$

then we have instead of (1.10)

$$c = \bar{c}_1 + \sum_{k=1}^{\infty} B_{2k} \exp\left(-\frac{\beta\xi}{2} - \lambda^2_k\tau\right) \sin \eta_k\xi \qquad (1.12)$$

where

$$\xi = 1 - h, \quad \bar{c}_1 = \frac{\alpha}{\beta}\xi + \left(1 - \frac{\alpha}{\beta} - \frac{\alpha}{\beta^2}\right), \quad \lambda^2_k = \eta^2_k + \frac{\beta^2}{4}$$

$$\tan \eta_k + \frac{2\eta c}{\beta} = 0, \quad \beta_{2k} = \frac{4\mu_k(a_k + b_k + d_k)}{(2\mu_k - \sin 2\mu_k)(\beta^2 + 4\mu_k^2)}$$

$$a_k = \frac{2\alpha}{\beta}\left[\beta \exp\frac{\beta}{2}\sin\mu_k - 2\mu_k \exp\beta/2 \cos\mu_k\right],$$

$$\delta_k = \frac{4\alpha}{\beta}\left[(\beta^2+4\mu_k^2)\exp\beta/2 \sin\mu_k + 4\beta\mu_k(1 - \exp\beta/2\cos\mu_k)\right],$$

$$d_k = 2\left(1 - \frac{\alpha}{\beta} - \frac{\alpha}{\beta^2}\right)(2\mu_k \cos\mu_k - \beta\sin\mu_k)\exp\beta/2$$

For an expending estuary of constant depth H we have from equation (1.3) with n = 1

$$\frac{\partial c}{\partial t} = D\frac{\partial^2 c}{\partial r^2} + \frac{\beta_1}{r}\frac{\partial c}{\partial r} - q, \quad \beta_1 = D - \frac{Q}{\varphi H} \tag{1.13}$$

where r is the instanteneous radius, Fig. 1. φ is the estuary expansion angle, and Q is the river flow.

By introducing dimensionless coordinates as in equation (1.5).

$$r = \xi L, \quad t = \tau L^2/D \tag{1.14}$$

equation (1.13) rearranges, with $q = 0$, into

$$\frac{\partial c}{\partial \tau} = \frac{\partial^2 c}{\partial \xi^2} + \frac{\gamma}{\xi}\frac{\partial c}{\partial \xi}, \quad \gamma = \beta_1/D \tag{1.15}$$

Under the boundary conditions

$$\tau = 0,\ c = 0,\ \xi = 0,\ c = 1;\ \xi = \infty,\ c = 0 \tag{1.16}$$

an automodel solution of equation (1.15) can be obtained in the form

$$c(\tau,\xi) = \Gamma(a, \xi^2/4\tau)/\Gamma(a) \tag{1.17}$$

where $\Gamma(a)$ is a gamma-function and $\Gamma(a,n)$ is an incomplete gamma function

$$u = \xi^2/4\tau, \quad a = \frac{1-\gamma}{2} > 0$$

A solution of (1.17) is represented in Fig. 2 with some values of a proportional to the river flow Q. The area where large amounts of fresh water are present is largely dependent on the parameter a. Thus with $c = 0.95$ the length of this zone increases 20-fold as a increses from 1 to 4.

For an expanding eatuary of finite length with a boundary condition on the right-hand side in the form

$$\xi = 1, \quad c = 0 \tag{1.18}$$

and other boundary conditions in the form (1.16), the distribution of the fresh water content $c(\xi,\tau)$ can be obtained as a Bessel series of functions

$$c = 1 - \xi^{2a} + \xi^{a}\sum_{k=1}^{\infty} A_k e^{-\lambda_k \tau} I_m(\lambda_k \xi) \tag{1.19}$$

where λ_k are roots of the equation $I_m(\lambda_k) = 0$,

$$A_k = -\frac{\lambda_k^{m-2}}{I^2_{m-1}(\lambda_k)2^{m-2}\Gamma(a)}$$

2. In weakly stratified estuaries the salinity is different in the surface stratum and deeper waters, with a noticeable transfer through medium depths. The horizontal and vertical salinity gradients exist both in the surface and deeper strata: salt is advectively transferred in surface strata towards the sea and in bottom strata towards tha land.

The chief subject of interest in the hydrodynamics of weakly stratified estyarues is the residual circulation in small, relatively narrow estuaries unaffected by tides. The estuary is viewed as small when the tidal wave is shorter than the estuary length but other short-term perturbations such as wind make an impact on the average circulation and mass transfer. Circulation fluxes in estuaries may be classified into lateral and vertical which both result from the action of the same externae and internal forces such as the wind, tides, topography, river flow and sea water salinity.

For narrow estuaries whose length is assumed to be much larger thab the width, the properties of lateral fluid and salt flows do not change much. Therefore three-dimensional equations of the eatuary hydrophysics can be integrated over the width and reduces to a simpler two-dimensional form. For an estuary, two-dimensional in the vertical plane, weakly stratified, and having a constant depth and width the equations of motin, continuity, and diffusion of a conservative solute averaged over the tide period for a stationary case have the form [3]

$$u = \frac{\partial u}{\partial x} + V \frac{\partial u}{\partial y} = - \frac{1}{\rho_o} \frac{\partial p}{\partial x} + \frac{\partial}{\partial y} (A_{xy} \frac{\partial u}{\partial y}), \qquad (2.1)$$

$$0 = - \frac{1}{\rho} \frac{\partial p}{\partial y} + g,$$

$$0 = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y}$$

$$u \frac{\partial c}{\partial x} + V \frac{\partial c}{\partial y} = \frac{\partial}{\partial x} \left(K_x \frac{\partial c}{\partial x}\right) + \frac{\partial}{\partial y} \left(K_y \frac{\partial c}{\partial y}\right)$$

where u and v are the liquid velocity componenets along the x and y axes, respectively; P and ρ are the liquid pressure and density; A_{xy} is the coefficient of turbulent viscosity; K_x and K_y are coefficients of turbulent diffusion; ρ_o is the density of fresh water, c is the salt content; and g is gravity.

The liquid state equation has the form

$$\rho = \rho_o (1 + 0.76c) \qquad (2.2)$$

Introducing a function such as Ψ (x, y) by the relations

$$u = \frac{\partial \Psi}{\partial y}, \quad v = -\frac{\partial \Psi}{\partial x} \tag{2.3}$$

we have boundary conditions

$$y = 0, \ \Psi = 0, \ \frac{\partial^2 \Psi}{\partial y^2} = \frac{\tau}{\rho_0 A_{xy}}, \ \frac{\partial c}{\partial y} = 0,$$

$$y = H, \ \Psi = Q, \ \frac{\partial \Psi}{\partial y} = 0, \ \frac{\partial c}{\partial y} = 0, \tag{2.4}$$

$$\lim c = 0, \ \lim \frac{\partial \Psi}{\partial x} = 0, \ x \to -\infty$$

where τ is the wind tension.

An approximate solution of equations (2.1) under the boundary conditions (2.4) can be obtained by the Shvets-Targ method [3]. It provides a fairly accurate description of a two-stratum fluid circulation in actual estuaries where the botton sea water moves upwards towards the land whereas the surface flux is sea-bound.

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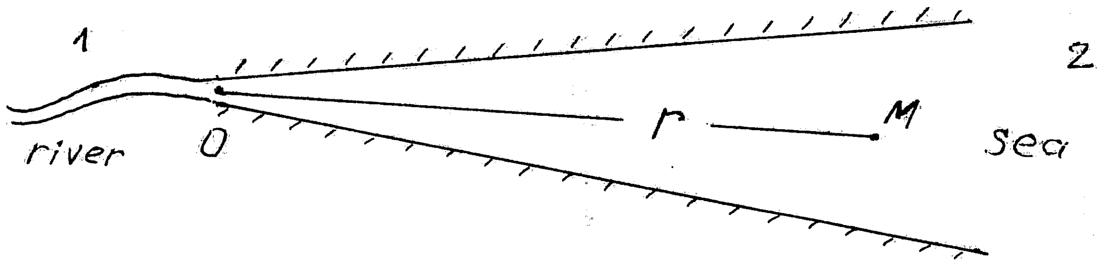

Fig. 1. Schematic diagram of an expanding estuary
1. river 2. sea

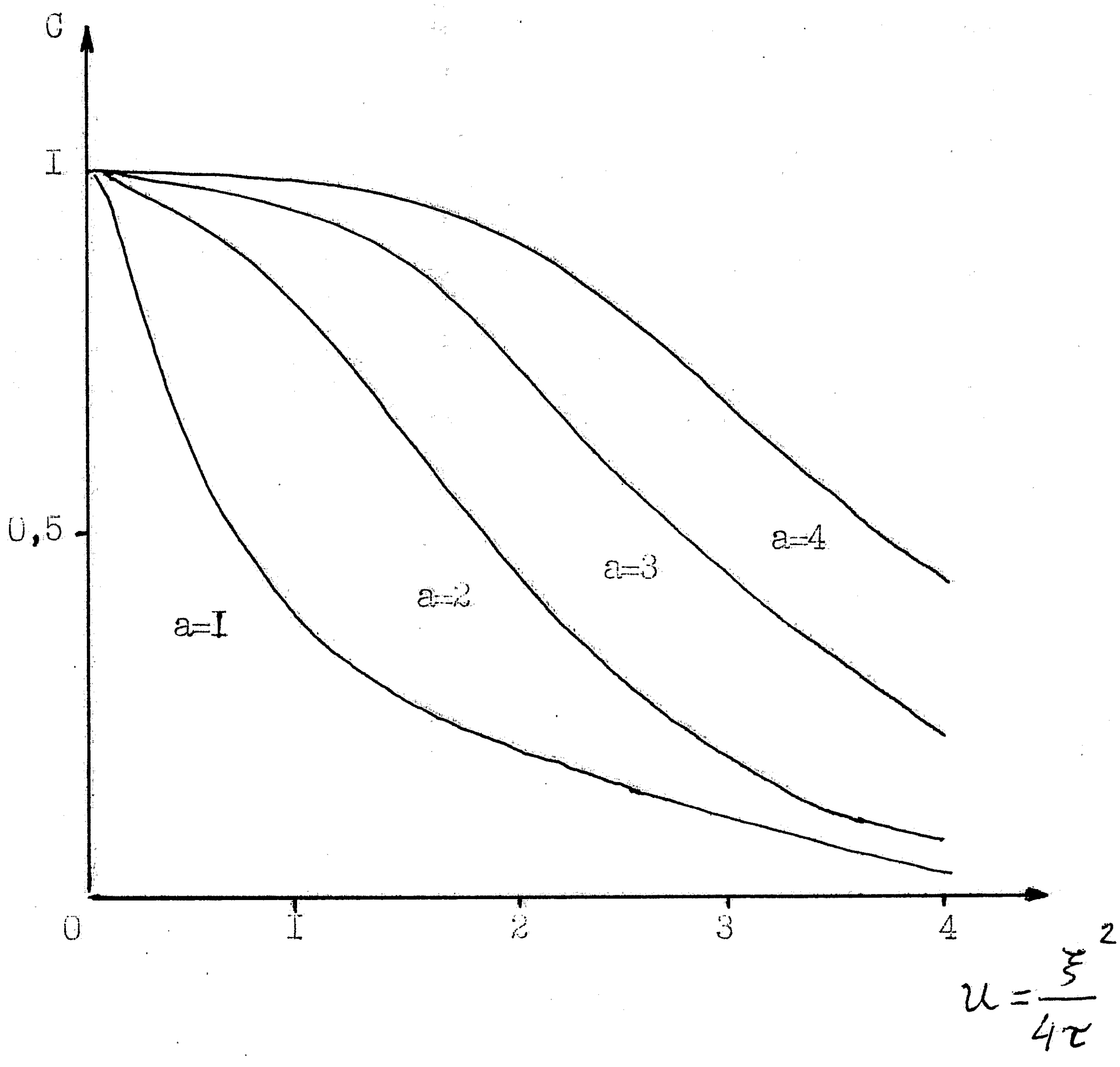

Fig. 2. The effect of river flow on the dynamics of sea water desalination by fresh water in an expanding estuary.

CAN THE DILUTION OF AN ACCIDENTALLY RELEASED POLLUTANT BE INCREASED BY REGULATING THE RIVER?

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INTRODUCTION

The aim of the paper is to discuss, whether one can influence the dilution of a harmful substance , that has accidentally been released into a river during a short, limited period of time. By regulating the flow in the river one could increase the the discharge to perhaps obtain an increase of the mixing process, and hence dilute the pollutant more effectively than without regulation. Conversely, a decrease in the discharge would decrease the transport velocity of the pollutant and perhaps decrease the dilution, but give more time for counteractions.

The above idea requires first of all that there is a regulating facility in the river (a gate, a hydropower station etc.) and sufficient storage of water to sustain an increased discharge for the required time period. The effect of possible operational manouvers can be studied with a mathematical model, which solves the appropriate transport and diffusion equations numerically. The diffusion coefficient of the pollutant has to be estimated with reasonable accuracy to make comparison of different regulatin alternatives meaningful.

An example of the behavior of a hypothetical accidental release in the Kymijoki River in Finland will be presented.

EQUATIONS

For the one-dimensional case the following transport-diffusion equation is used:

$$\frac{\partial (AC)}{\partial t} + U \frac{\partial (AC)}{\partial x} = \frac{\partial}{\partial x}\left(AK \frac{\partial C}{\partial x}\right) \qquad (1)$$

where	U=Q/A=	mean cross sectional flow velocity
	A=	cross sectional area
	Q=	discharge
	K=	dispersion coefficient
	t=	time
	x=	longitudinal coordinate

This equation is usally split into one transport equation

$$\frac{\partial (AC)}{\partial t} + U \frac{\partial (AC)}{\partial x} = 0 \qquad (2a)$$

and one diffusion equation

$$\frac{\partial(AC)}{\partial t} = \frac{\partial}{\partial x}\left(AK\frac{\partial C}{\partial x}\right) \qquad (2b)$$

These two equations are then solved numerically in two separate steps.

It is well known (Cunge & al, 1980) that considerable numerical diffusion, appart from the true physical diffusion, can occur when solving the transport equation, eq. 2a. Sometimes the numerical diffusion can be even greater than the physical diffusion. In our case it is important, that the numerical diffusion is kept insignificant, because the main interest is to study the change in (physical) diffusion of the pollutant as a result of changes in flow conditions. Numerical diffusion would destroy the validity of the comparison between different discharge alternatives.

The Holly-Preissmann 'two-point fourth-order scheme' (Holly & Preissmann 1977) for the transport equation is sufficiently accurate for our purpose. In this scheme not only the concentrations are convected, but also the concentration gradients, whereby it is possible to obtain fourth-order accuracy using only two gridpoints at a time.
This scheme was adopted in the current case.

There exists a number of schemes to solve the diffusion equation, eq. 2b. Here an explicit scheme by Chevereau and Preissman was used (cf. Cunge & al., 1980). Now it only remains to determine the diffusion coefficient K.

Elder (1959) proposed:

$$K = 5.93\ D\ U_* \qquad (3a)$$

D = mean depth
U_* = shear velocity

McQuivey and Keefer (1974):

$$K = 0.058\ Q\ /(S\ B) \qquad (3b)$$

S = energy slope of water level
B = channel width

Fischer (1973):

$$K = 0.011\ U^2\ B^2\ /(D\ U_*) \qquad (3c)$$

An indication of the effects of an increase of the discharge can be obtained analytically. If, in the case of uniform flow, the discharge is doubled, it can be shown that with eqs. 3a and 3b K

will also be doubled, and that with eq. 3c K will decrease. Because the flow velocity will increase with a factor of only 1.26, an overall increase in the dilution is to be expected in the first two cases. In the third case there will theoretically be a decrease in the rate of dilution. It can be concluded that already the choice of formula for computing the dispersion coefficient greatly affects the results.
But the flow in natural rivers is not uniform, so no definite conclusions of the dilution can be made.

In the present case the formulation of McQuivey and Keefer(1974) was used, with the exception that the factor 0.058/S is kept constant for the whole reach.

HYPOTHETICAL ACCIDENTAL RELEASE OF A POLLUTANT IN THE KYMIJOKI RIVER

The simulated reach of the Kymijoki River is about 75 km long and includes several hydropower stations. Along the river there ~~is~~ also many pulp factories, from which accidental releases of sodium hydroxide have ~~happened~~. The river is quite irregular and was described by 290 cross sections with a distance of 84...1220 m. The dispersion coefficient was calculated as K= 25 Q/B, obtained from a field test, giving local values of K=12...150 m^2/s.

It was assumed, that an accidental release of a pollutant happened at location x=103.6 km (x= distance from the sea) at time t=0, it reaches its maximum after two hours and goes to zero at t=4 hours-.Three cases were simulated: a) the discharge was kept constant (Q=200 m^3/s), b) the discharge was increased to 400 m^3/s at t= 5 hours, and c) the discharge was 400 m^3/s from the start.

The value of the flow velocity (or discharge) needed to solve the transport equation (eq. 2b) was obtained from a one-dimensional mathematical unsteady flow model using the same cross sections.

It was found that the dilution is quite strong during the first 10 km of the simulated reach. It could also be concluded, that increasing the discharge increased the dilution with 11 % 76.8 km downstream the point of release. The difference in dilution between the three cases cannot be considered of practical importance, though.

The transport velocity was increased by about 80 %, when doubling the discharge.

The numerical diffusion was found to be very much smaller than the physical one.

DISCUSSION

Increasing the discharge after an accidental release of a pollutant will result in a slight increase of the dilution, when using the eq. 3b to calculate the local value of the diffusion coefficient. The arrival time of the peak concentration was affected to a greater extent.

It was found, that some instability appeared in the Holly-Preissmann scheme if the cross sections were to far appart. Holly and Preissmann (1977) recommend, that the pollution cloud should be described with at least ten points, in order to make the calculation accurate. In the present case the limited extent of the pollution cloud in the beginning of the simulation period requires a short distance between cross sections in the beginning of the river reach, before the cloud is stretched out due to mixing. This restriction could be removed by using some other computational method in the beginning of the simulation,e.g the 'small cloud method' used by SOGREAH (Holly and Usseglio-Polatera, 1984).

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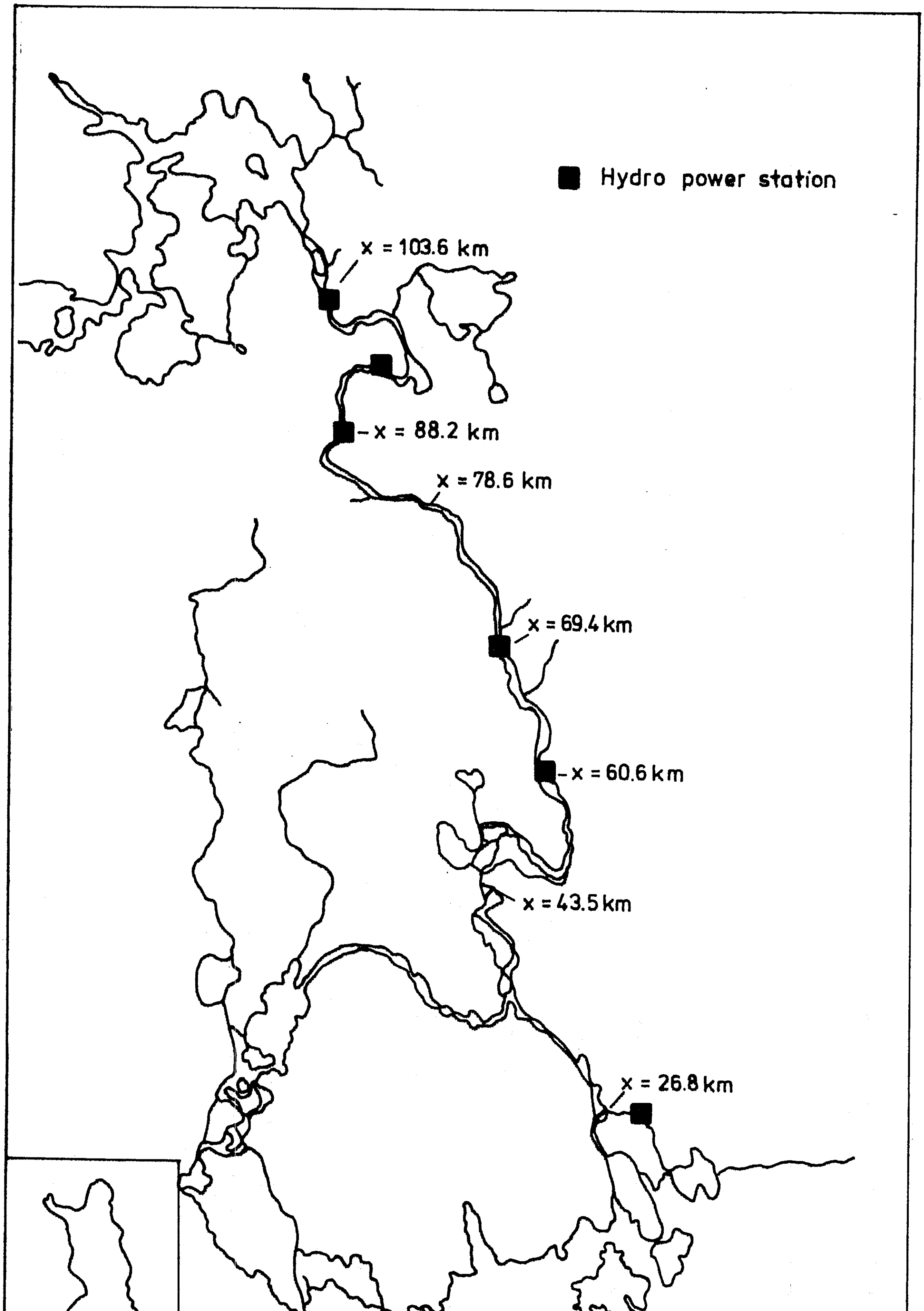

Fig. 1. The Kymijoki River

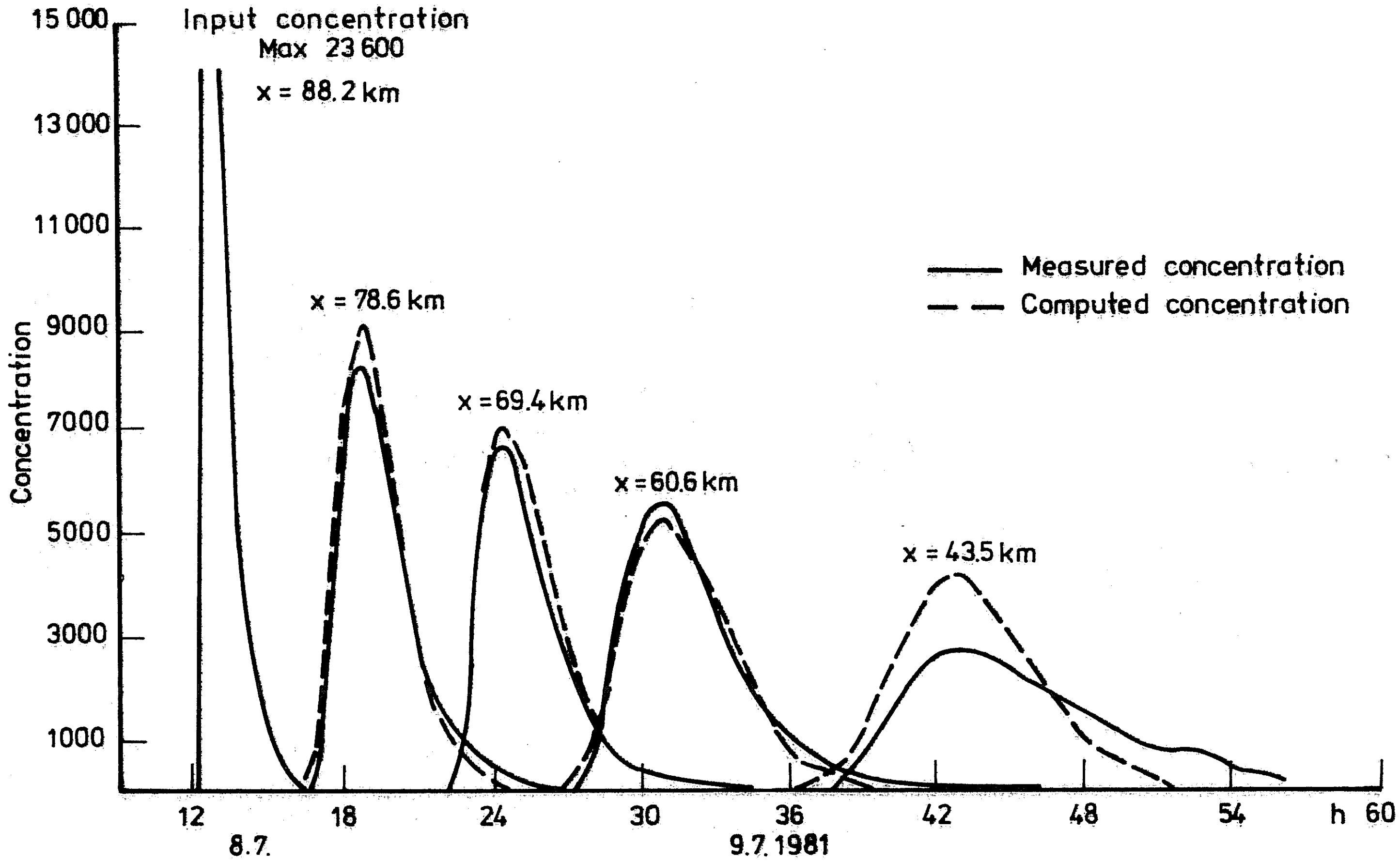

Fig. 2. Measuered and computed concentrations in Kymijoki River during field experiment.

x = longitudinal coordinate beginning from river mouth.

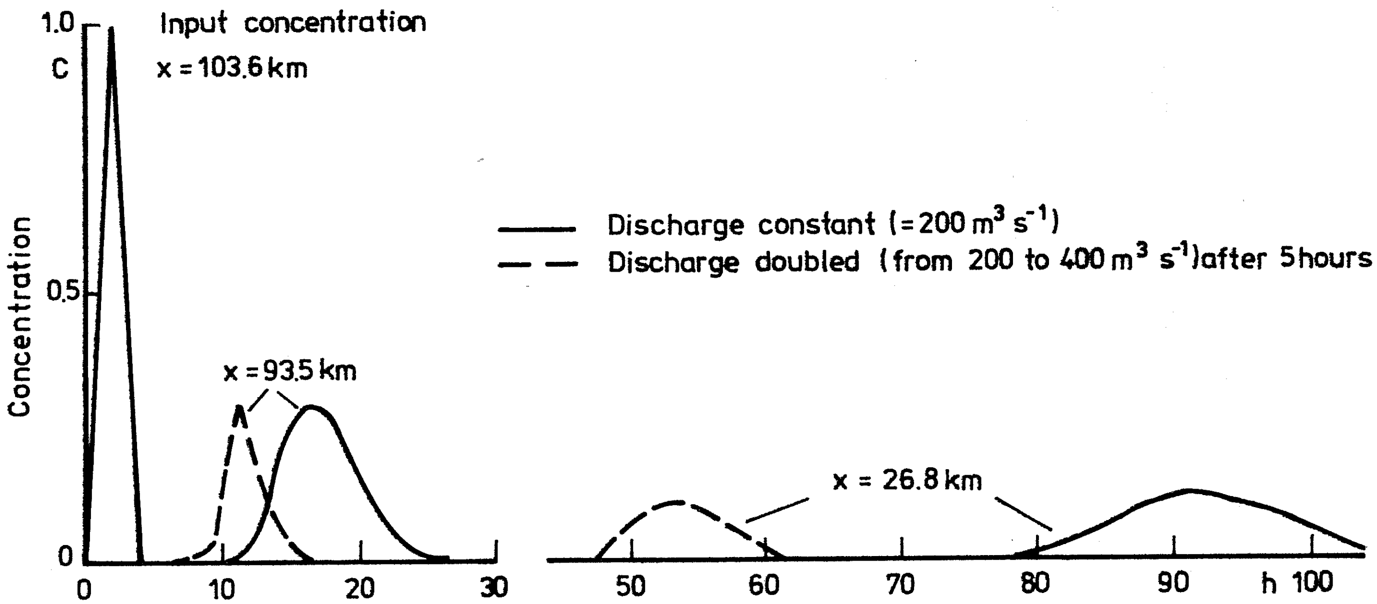

Fig. 3. Dilution of an accidental release of a pollutant (hypothetical case).

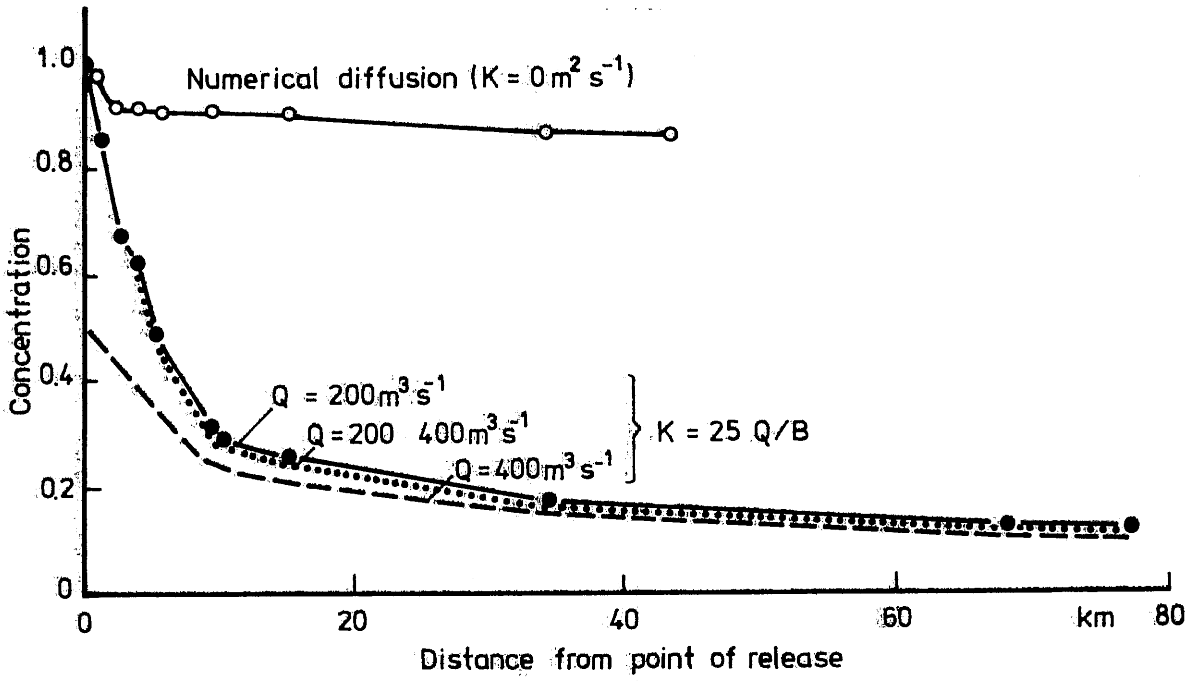

Fig. 4. Decrease of peak concentration.

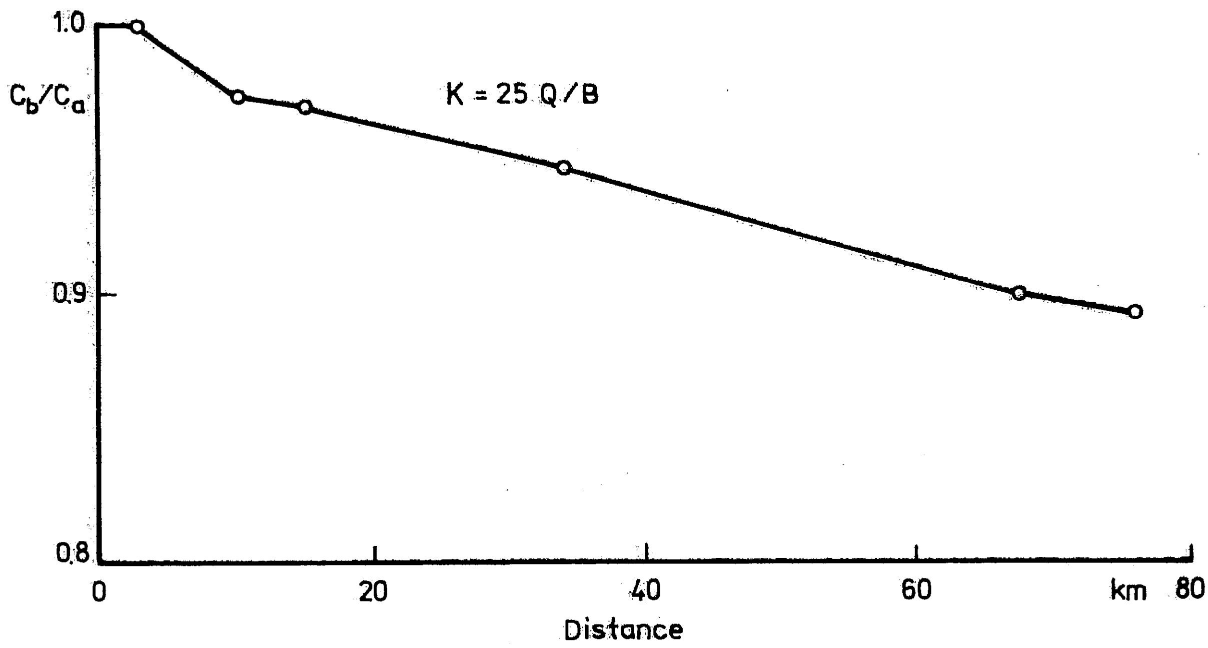

Fig. 5. Comparison of dilution. C_a= peak conc. when Q= 200 m^3/s, C_b= peak conc. when Q= 200--> 400 m^3/s.

EXAMINATION OF MODEL ADEQUACY AND ANALYSIS OF PHOSPHORUS DYNAMICS IN LAKE KUORTANEENJÄRVI - A CASE STUDY WITH TWO LAKE MODELS *)

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The dynamics of P and algae of Lake Kuortaneenjärvi, Finland, were studied. Two mathematical models were used to analyze the lake behavior. One was tailored for the application with special emphasis on analysis of algal dynamics. The other focuses on P transformations and was transferred with slight recalibration from Lake Balaton, Hungary and Ivankovskoe Reservoir, USSR. A subtask was hence to examine and discuss the adequacy of complex ecological lake models in a case of inadequate field data. The results showed that incoming P fractions differed greatly between the two basins of the lake. Organic fractions were more dominating in the lower basin. Net sedimentation was only 20 % of the gross sedimentation. The calibration results of the two models were rather adequate as far as the observed variables were considered but the overparametrization of the models came out in inadequacies of unobserved items. The original scope of the modeling comprised also evidently to the adequacy.

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*) The paper will be published elsewhere

ON THE ADEQUACY OF LARGE-SCALE MODELS IDENTIFIED WITH INCOMPLETE FIELD DATA - A CASE STUDY WITH TWO LAKES MODELS*)

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ABSTRACT

This study was motivated by the question: "Within a large-scale model describing a complex natural system, how adequate and valid can the items identified with incomplete field data be assumed to be in comparison with subsystems better observed?". This question was studied by applying and comparing two water quality models against the same set of field data from Lake Kuortaneenjärvi, Finland.

One of the models has previously been constructed to describe algal dynamics based on N and P cycles of Lake Kuortaneenjärvi. The other has been developed for the analysis of phosphorus dynamics and transformations of Lake Balaton, Hungary.

Nine model variables were chosen to represent the models and different categories of observation types: directly and indirectly observed as well as non-observed variables. Statistical comparisons were made between computed and observed values as well as between the simulated values obtained with the two models.

The results show distinctly the problem of uncertainty and inadequacy related to modeling of non-observed subsystems e.g. transferring submodels or complete models identified earlier with data from some other systems.

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